ARIZONA DEPARTMENT OF WATER RESOURCES
DAM SAFETY SECTION

State Standard

for

Floodplain Hydraulic Modeling

Under the authority outlined in ARS 48-3605(A) the Director of the Arizona Department of Water Resources establishes the following standard for Floodplain Hydraulic Modeling in Arizona.

The guidelines outlined in State Standard Attachment 9-02 entitled "Floodplain Hydraulic Modeling," or an alternative procedure reviewed and accepted by the Director, will be used in modeling floodplains for fulfilling the requirements of Flood Insurance Studies and local community and county flood damage prevention ordinances.

Floodplain hydraulic modeling standards will apply to all watercourses identified by the Federal Emergency Management Agency as part of the National Flood Insurance Program, all watercourses that have been identified by local floodplain administrators as having significant potential flood hazards and all watercourses with drainage areas more than 1/4 square mile or a 100-year discharge estimate of more than 500 cubic feet per second. Application of these guidelines will not be necessary if the local community or county has in effect a drainage, grading or stormwater ordinance that, in the opinion of the Department, results in the same or greater level of flood protection as application of these guidelines.

This requirement is effective July 30, 2002.

Copies of this State Standard and State Standard Attachment can be obtained by contacting the Department's Dam Safety Section at (602) 417-2445.

ARIZONA DEPARTMENT OF WATER RESOURCES
DAM SAFETY SECTION

Floodplain Hydraulic Modeling

500 North Third Street
Phoenix, Arizona 85004

(602) 417-2445

DISCLAIMER OF LIABILITY

The Arizona Department of Water Resources is not responsible for the application of the methods outlined in this publication and accepts no liability for their use. Sound engineering judgment is recommended in all cases.

The Arizona Department of Water Resources reserves the right to modify, update, or otherwise revise this document. Questions regarding information contained in this document and/or floodplain management should be directed to the local floodplain administrator or the office below:

Dam Safety Section
Arizona Department of Water Resources
500 North Third Street
Phoenix, Arizona 85004

Phone: 602-417-2445
FAX: 602-417-2423

TABLE OF CONTENTS

Arizona State Standard Attachment on Floodplain Hydraulic Modeling

1. Introduction

1.1 Purpose and Background

The purpose of this document is to provide guidance on mathematical modeling of hydraulic processes in watercourses and floodplains. This type of modeling is often the basis for determining floodplain limits for given flow events (e.g., 1% chance or 100-year discharge) or the impact of a proposed project on water surface elevations and floodplain limits. Modeling procedures and techniques can greatly affect computed water surface elevations and floodplain limits for a given flow rate. This Standard was prepared to help identify proper mathematical modeling practices and should be utilized for floodplain hydraulic modeling when preparing a floodplain modeling report.

Preparation of this Standard was carried out in three phases. Phase I consisted of a comprehensive literature search, data collection, and review. Phase II of the study consisted of review and evaluation of publications related to floodplain hydraulic modeling, review of one-dimensional hydraulic models, review of floodway encroachment methods, and the review and evaluation of three case studies. The items addressed in the case study evaluations included data requirements, input parameters, and modeling techniques. Phase III involved development of this Standard.

1.2 General

Mathematical modeling, in the context of hydraulic engineering, refers to predicting the state of a watercourse for any given flow rate based on theoretical and empirical relationships. These relationships are expressed in a series of mathematical equations, which are usually discretized, for use within a computational program. In the last few decades, mathematical modeling has become an accepted engineering tool.

The assumption made in this Standard is that the user will be preparing a hydraulic model of a watercourse to replicate (for an historic event) or predict (for a future or design event) hydraulic parameters such as water surface elevation, wetted top width, velocity and depth at certain locations along the watercourse. This Standard focuses on one of the most common types of models employed in practice: the one-dimensional model.

1.3 Overview

This State Standard Attachment is divided into the following chapters:

1. Introduction: This chapter, including purpose and background of this document.
2. One-dimensional Hydraulic Models: Types, names, availability, abilities, limitations, and data requirements of models.
3. Channel Geometry: Mapping, Cross Sections
4. Inflows and Outflows: Local inflows and outflows, tributaries, distributaries, and breakouts

5. Special Topics: Modeling of structures, selection of roughness values, floodway methods, supercritical and mixed flow
6. Floodway Methods: Floodway development, modeling procedure, “no-rise” condition, cumulative effects, generation of additional cross sections, reducing roughness, and comparison of conveyances
7. Good Modeling Practice: Documentation, common errors, quality control, example problems
8. References and Bibliography

In addition, appendices are included which provide supplemental information.

1.4 Related State Standards

Several of the State Standards produced to date are directly or indirectly related to floodplain modeling and/or floodplain mapping. These include:

SS1-97, Flood Study Technical Documentation
SS2-96, Delineation of Floodplains and Floodways in Arizona
SS3-94, Supercritical Flow
SS4-95, Identification of and Development within Sheet Flow Areas
SS5-96, Watercourse System Sediment Balance
SS6-96, Development of Individual Residential Lots within Floodprone Areas
SS7-98, Watercourse Bank Stabilization
SS8-99, Stormwater Detention/Retention

2. One-dimensional Hydraulic Models

2.1 General

This chapter provides background on one-dimensional (1-D) hydraulic modeling that will assist the reader in selecting the most appropriate model(s) for a given situation. As part of preparing this Standard, several 1-D models were reviewed. The 1-D models were divided into two groups: steady and unsteady flow models. Each model was listed in a matrix (Appendix A), and evaluated in thirty categories. Use of the matrix will provide guidance for the applicability and appropriateness of certain models to a given watercourse and its associated floodplain. Several of the models were used in evaluation of case studies and are described in greater detail later in this attachment.

2.2 Definition

In general, three physical coordinate dimensions are necessary to describe the properties of a flow field (three-dimensional or 3-D flow). For some cases, there is very little or no change in one coordinate direction, and two coordinate dimensions are sufficient to describe the flow field. For example, flow in bays and estuaries can often be modeled using two-dimensional (2-D) flow in the horizontal plane with the assumption that the detailed vertical velocity gradient is not important to the study and can be approximated by empirical equations.

The simplest flow field is the 1-D case, in which only one coordinate is needed to describe the flow field. An example is flow in a conduit in which the velocity is constant vertically and horizontally at each section but varies with distance along the conduit. In actuality, the velocity is never constant across a conduit section; however, for problems in which we are primarily interested in velocity parallel to the direction of the conduit, we can describe the flow as 1-D with that one dimension parallel to the conduit. Although flow in natural watercourses is never truly 1-D, for many cases this simplification produces acceptable results in predicted hydraulic parameters. Because of the general applicability of 1-D models and their ease of use, they have become the standard for many applications. However, the reader is warned that for cases where flow can clearly not be described by one dimension (e.g., spreading flows on alluvial fans or unstable alluvial channels), a 2-D or even 3-D model might be needed to accurately model the flow field.

2.3 Model Selection Considerations

Generally, before beginning a modeling study, a site visit should be conducted to understand the nature of the problem and the purpose of the study. In some instances, the model to be used for a study is specified by regulatory or reviewing agencies. However, the choice of the model to be used often lies with the modeler. Given the number of existing models, it is necessary to determine which model is best suited for a specific project. Some of the most important questions to consider are addressed in the following sections.

2.3.1 Steady versus Unsteady Flow

1-D models can be subdivided into unsteady and steady flow models. Simply put, unsteady flow models consider the effects of time (and the resultant change in the rate of water storage) while steady flow models do not. For example, unsteady flow models consider the variation of flow with time (e.g., a hydrograph at a point in the watercourse) within a watercourse reach whereas steady flow models use only a single flow rate (with no associated time) in the same reach. As a result, unsteady flow models will take into account the effects of a limited volume of water for a flow event (i.e., the area under a hydrograph), while the steady flow model considers the event having essentially an infinite volume of water. Unsteady flow models can also simulate changes in boundary conditions with time (e.g., the opening and closing of a gate) while steady flow models cannot. If there is concern that timing and volume effects will be significant for your project, an unsteady model should be considered.

Take for instance a project consisting of a small watercourse with a wide floodplain and a "flashy" hydrograph (i.e., quickly rising and falling flow rates). Assuming that the system can still be modeled as 1-D flow, an unsteady flow model will simulate flow leaving the channel, entering the overbank area(s) and later returning to the channel further downstream. The maximum overbank inundated area may be limited by the amount of water in the flood hydrograph and the duration of the spill into the floodplain. In addition, this spillage may reduce the peak discharge entering the downstream reach. However, if the project is simulated with a steady flow model using the peak flow rate throughout the reach, the overbank inundation area will be filled to its maximum capacity since there is no limitation on the volume. In addition, the peak discharge entering the downstream reach is assumed to be the same as the upstream reach.

Why not always use an unsteady model then? The principal reasons are:

1) Unsteady flow models are usually more complex than steady flow models, and therefore more costly.
2) High water marks, often used for calibration/confirmation of hydraulic models, give only the maximum water surface elevation with no indication of the timing when the maximum was reached.
3) Natural hydrographs with gradually changing flow rates can be well approximated by a peak flow assumption.
4) Often, a complete flow hydrograph is not available or cannot be easily developed. However, a peak flow rate can often be assumed or computed.
5) Steady flow models provide a more conservative estimate of inundated floodplain areas because of the volume considerations discussed.

Steady flow models continue to be used most often for channel and floodplain studies. Consideration of the importance of timing and volume effects for a project will govern whether an unsteady flow model should be used.

2.3.2 Abilities and Limitations of Common Models

Many 1-D models are available to the practitioner. Appendix A lists some of the more commonly used models. Some models were developed with a particular purpose (e.g., simulate flow through bridges) and thus will be strong in a particular application but weaker for other applications. Appendix A also lists the computational theory, strengths, and weaknesses of many of the more popular models.

2.3.3 Backwater (Multiple Cross Section) versus Single Cross Section Models

Some of the models listed in Appendix A utilize multiple contiguous cross sections to simulate flow along the watercourse, while other models use a single "representative" cross section. The former group of models provides an estimate of how hydraulic properties will change along the watercourse due to changes in longitudinal geometry, roughness, structures, etc., in adjacent cross sections. The latter group uses a normal depth approximation to determine hydraulic parameters at a single cross section, ignoring any hydraulic effects of adjacent cross sections. In general, the user must enter the cross section geometry, roughness, and an energy slope for these models. The user must also specify either the discharge or flow depth, and the program will solve for the other variable.

Multiple cross section (or "backwater") models will provide more accurate results. Single cross section models are useful for reaches where a great level of detail is not needed and/or there is not a great deal of topographic information from which to develop cross sections. Because of the normal depth approximation, these single cross section models cannot simulate backwater effects (e.g., upstream of a structure under subcritical flow) or drawdown areas (e.g., immediately upstream of a weir). However, single cross section models can be useful for reconnaissance level studies and approximate floodplain mapping (e.g., Flood Insurance Rate Map A zones).

2.3.4 Acceptance and Availability of Models

The choice of a model should be based on the considerations listed in the preceding sections. However, some governmental agencies have a preferred model or models, especially if that agency has developed models that suit its purposes (e.g., the U.S. Army Corps of Engineers and the HEC models). For flood insurance studies, FEMA is the regulatory agency for the National Flood Insurance Program (NFIP). FEMA publishes a list of accepted hydraulic models, which may be accessed from FEMA's web site (http://www.fema.gov/mit/tsd/en_hydra.htm). The list of hydraulic models accepted by FEMA (dated January 2002) is provided in Appendix B.

The availability of the computer programs listed in Appendices A and B varies widely. Programs developed by the U.S. Federal Government are Public Domain Software and can usually be obtained at little or no charge. Some agencies let users download the software for free from the agency's web site. Information about downloading the U.S. Army Corps of Engineers' HEC models is at http://www.hec.usace.army.mil/software/software_distrib/index.html. Third-party developers sometimes offer enhanced versions of the Government software and will therefore charge for their product. Some agencies, such as the U.S. Army Corps of Engineers, have established official vendors of their software who sell and offer technical support to private sector users. Other programs on these lists have been partially or completely developed by private companies and/or individuals who sell their product via a variety of methods.

2.4 1-D Model Limitations

The modeler must consider to what extent natural watercourse flow can be modeled without violating the basic concepts and assumptions of the 1-D flow equations. The de St. Venant equations for unsteady flow are based upon the following assumptions (Cunge et al., 1980):

1) The flow is approximately one-dimensional, i.e., the velocity is nearly uniform over the cross section and the water level across the section is horizontal.

2) The streamline curvature is small and vertical accelerations are negligible, hence the pressure is hydrostatic.
3) The effects of boundary friction and turbulence can be accounted for through resistance laws analogous to those used for steady state flow.
4) The average channel bed slope is small so that the cosine of the angle it makes with the horizontal may be replaced by unity.

Because only two dependent variables are sufficient to describe 1-D flow, we need only two equations, each of which must represent a physical law. However, we can formulate three physical laws in such flow: conservation of mass, momentum, and energy. When the flow variables are not continuous (i.e., discontinuities of the water surface profile such as at hydraulic jumps and other rapidly varied flow situations), two representations are possible: conservation of mass and momentum, or conservation of mass and energy. The two representations are not equivalent, and only one is correct, depending on the hydraulic phenomenon.

When the flow variables are continuous, either of the representations may be used, and they are equivalent. The mass-momentum couple of conservation laws are applicable to both discontinuous and continuous situations where the mass-energy couple is not. However, many 1-D models solve the energy equation via either the standard step or direct step methods. Unless the particular model has a momentum solution to discontinuous or rapidly varied flows, the model may give inaccurate results obtained from solving the energy equation. It is also noted that 1-D models developed by governmental agencies or well-known private industry and/or third-party vendors typically contain “flags” that help to prevent the user from violating mass-energy laws when performing hydraulic modeling.

The reader should be aware of the limitations of 1-D models based upon the assumptions listed above. If there are significant violations of these assumptions, an assessment of the possible errors in the results should be made. In turn, if these possible errors are not acceptable, either very conservative 1-D modeling assumptions must be made or a 2- or 3-D model considered.

2.5 Data Requirements and Collection

All 1-D models require some representation of channel geometry, most often represented by cross sections at various locations along the watercourse. Most often, the modeler supplies channel discharge at the cross sections and the model will solve for flow depths and velocities. For some single cross section models, the modeler supplies the energy slope and water depth and the model will solve for the discharge. Other single cross section models require the user to specify, in addition to the geometry, three of the following four variables and the model will solve for the unknown variable: roughness, slope, discharge, water depth.

For multiple section models, the modeler must also supply reach boundary conditions. The most usual case is to supply a known water surface (downstream for subcritical flows and upstream for supercritical flows) and a flow rate at the upstream end of a study reach. If the boundary water surface is unknown, it is sometimes computed as either critical depth or normal depth (an energy slope must also be supplied in this case). Unsteady flow models will allow the boundary conditions to vary with time, while boundary conditions are fixed for a steady flow simulation.

3. Channel Geometry

3.1 General

Channel geometry is a common input for all hydraulic models. The necessary level of accuracy depends on the purposes of a particular study. In addition, special considerations must be taken with 1-D models to obtain satisfactory results when simulating processes occurring in a 4-dimensional world (3-D plus time).

3.2 Mapping Requirements for Hydraulic Modeling and Floodplain Delineation

Mapping in this Standard refers to the gathering of topographic information for an area of interest. The goal of mapping activities, as related to 1-D hydraulic modeling, is to obtain topographic information in sufficient detail such that the resulting discrete cross sections will reflect the geometric surface characteristics of the watercourse (i.e., what the water "sees"). The level of topographic detail needed for a particular study will be related to the type of terrain being modeled, and the level of detail needed in the results.

Cross sections are usually obtained in one of two ways: either by direct field surveys, or by "cutting" from contour maps (either on paper or in electronic format). Regardless of which method is used, the guidance given in Section 3.3 for locating cross sections should be employed. Direct surveys tied into accepted benchmarks yield very accurate results, but can become economically unfeasible when very long reaches of watercourse or very wide floodplains are being modeled. Photogrammetric mapping methods are commonly used in these cases. Because photogrammetric methods will not identify any submerged channel geometry, the data must often be supplemented with hydrographic survey data. In Arizona, however, many watercourses are dry for most of the year, which eliminates this problem.

For wide floodplains, direct field surveys may be less accurate than photogrammetric methods because correct cross section orientation within wide braided overbank areas cannot be ascertained as well in the field. An advantage to photogrammetric methods is the ability to "cut" new cross sections or re-orient them (as needed for hydraulic modeling considerations) without having to conduct a new field survey. Industry standard photogrammetry controls need to be applied to achieve sufficient accuracy for a chosen contour interval. Note: In this regard, keep in mind that topographic mapping obtained using photogrammetric methods is generally only accurate to within ± ½ of a contour interval.

3.2.1 Flood Insurance Studies

For flooding sources to be studied in detail, FEMA normally requires a 4-foot contour mapping (FEMA, 1995). However, field surveys may be used in place of or in addition to the topographic mapping. Vertical error tolerance for field surveys must be within ±0.5 foot across the 100-year floodplain (FEMA, 1995). FEMA has also produced guidelines for photogrammetric mapping and surveying, which may be found in FEMA Publication No. 37 (FEMA, 1995). Many agencies maintain more stringent mapping requirements than FEMA. For example, the Flood Control District of Maricopa County requires 2-foot contours for floodplain maps (FCDMC, 2000). Therefore, the modeler should be aware of local agency requirements before any information is gathered.

For approximate floodplain studies, “all cross sections should be obtained from existing topographic maps” (FEMA, 1995). In many cases, the only available maps are the U.S. Geologic Survey topographic maps that often have contour intervals of 10 feet or more. Occasionally, local or state agencies may possess more detailed mapping information.

3.3 Cross Sections

Cross sections are the “backbone” of most hydraulic models. This section elaborates on the location, alignment, modification and interpolation of cross sections.

3.3.1 Location, Alignment, Configuration, and Spacing

Flowlines/streamlines should be sketched for the modeled watercourse reach for bankfull and flood discharges (or largest event to be analyzed). This can be accomplished by drawing on mylar or other transparent film overlain on physical maps of the study reach, or by drawing on electronic maps. If there are radical changes in flow area and direction between these two discharges, you may want to add an intermediate discharge. The flowlines should reflect expected contraction (nominally a 1 to 1 ratio) and expansion ratios (4 or 3 to 1 as a general rule).

Cross sections should be aligned such that they are perpendicular to the flowlines over their entire length (see Figure 3.1). If a series of discharges is to be modeled, cross sections will need to account for the changes in the flowlines. For significant changes in flow patterns, separate geometries may need to be created for the different discharges. The modeler must identify those areas that contain obstructions to the flow. If these obstructions cover a significant portion of the projected flow area (length perpendicular to the flowlines), cross sections must be inserted at frequent enough intervals at these locations to account for the effects of such obstructions.

Note: Cross sections should be placed in the influence zones upstream and downstream of these obstructions similar to cross sections 1 and 4 when modeling bridges (addressed in Chapter 5 and shown in Figure 5.1). Cross sections should be located, again, using the 1 to 1 contraction ratio and 4 or 3 to 1 expansion ratio as rules of thumb. Care should be taken to apply appropriate contraction and expansion coefficients at these locations. Additional guidance for the ratios and coefficients based on field and 2-D model data for bridges (USACE, 1995) is presented in Chapter 5.

Sometimes one is presented with cross sections obtained by others that are not aligned perpendicular to the expected flow lines. If the overall cross section is skewed more than 18 degrees from the perpendicular of the flow line, either the cross section needs to be resurveyed or reduced by an appropriate multiplier to obtain the projected flow area of the cross section.

Each cross section in a model is assumed to be representative of the geometry half way to the next cross section in both upstream and downstream directions. Cross sections should therefore be located at places such that they fully describe each segment of the reach geometry. Items to consider are changes in channel geometry, discharge, slope, roughness, and distance between cross sections for computational stability. Because changes occur closer together in smaller streams when compared to larger rivers, cross sections will need to be more closely spaced.

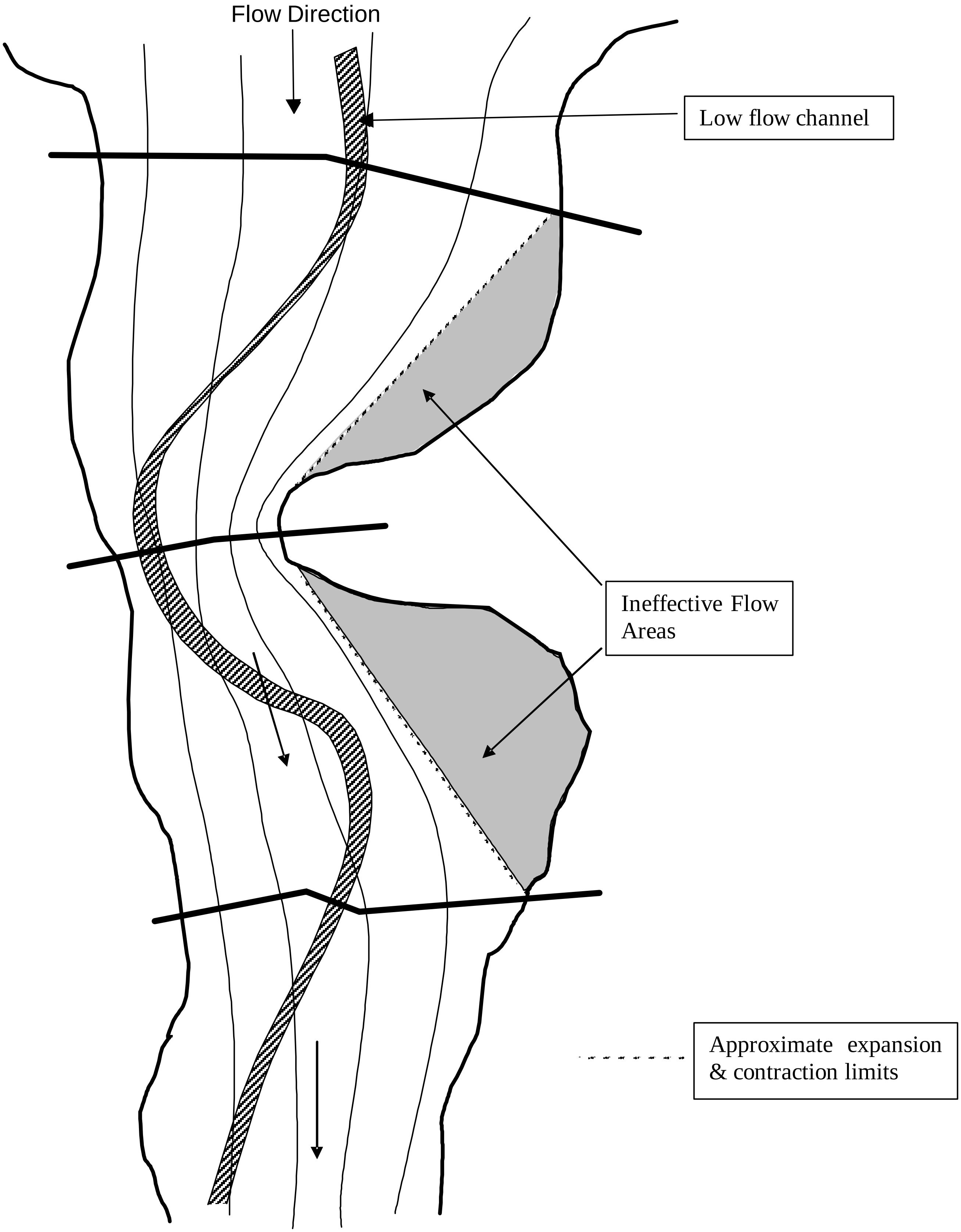

Figure 3.1. Streamlines and Cross Section Locations

An example of the proper use of representative cross section locations is a road that crosses the floodplain along the orientation of the cross section (with an associated Manning's "n" of 0.02). If there were adjacent natural cross sections (with appropriate "n" values) 500 feet upstream and downstream, the hydraulic model would simulate a 500-foot wide road since the "n" value of the cross section containing the road is assumed to have an influence half way to the upstream and downstream bounding cross sections.

The proper way to model such a situation is to bound the road with cross sections. Natural cross sections should be placed a nominal distance (e.g., one foot) upstream and downstream of the road (cross sections 1 and 4 in Figure 3.2) such that energy losses in the reaches approaching and leaving the road are correctly modeled. Two additional cross sections should be placed on the edge of the road (cross sections 2 and 3 in Figure 3.2) such that energy losses in the short reach over the road are correctly simulated. Cross sections 1 and 4 are the natural sections (e.g., appropriate natural "n" value), while cross sections 2 and 3 would have the roughness values of the roadway, 0.02.

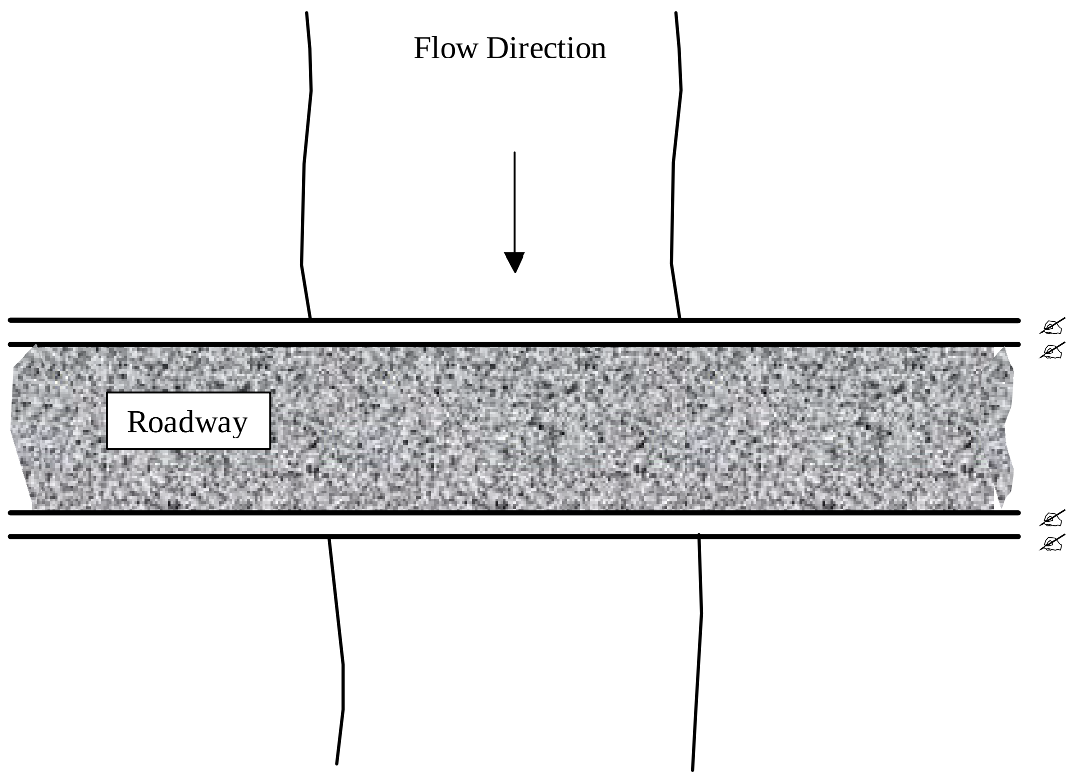

Figure 3.2. Proper Use of Representative Cross Sections

An alternate method to model this situation would be to locate a single cross section in the middle of the road (with the appropriate roughness), one natural section a road width upstream, and an additional cross section one road width downstream. Because the "n" value associated with the road has an influence of half way to each bounding cross section, the total effective road width will be equal to the actual width. Whichever method is chosen, the modeler should not forget to make the appropriate adjustments to the reach lengths due to the addition of new cross sections.

3.3.2 Addition of Cross Sections When Analyzing Proposed Conditions

When the modeler is asked to evaluate the impact of a project, a comparison of the pre- and post-project conditions is required. The project features often require adding cross sections to adequately model the project conditions. For instance, adding a bridge in HEC-RAS (discussed in detail later) requires the addition of six cross sections to fully model bridge hydraulics. These include the four cross sections input by the user and the actual bridge geometry at the downstream and upstream faces. For proper comparison of pre- and post-project conditions, the same number of cross sections must be in both models. The cross sections in the proposed conditions model must be added to the pre-project model; however, these cross sections should not contain any of the bridge features or influences of the bridge.

3.3.3 Subdivision and Channel Bank Locations

Many models require the location of bank stations that separate the main channel area from the overbank or floodplain areas of the cross section. Sometimes it is hard to delineate the channel from the overbanks. A good rule of thumb is to partition the cross section into areas of similar "n" values and then determine the channel and overbank limits. Note: For alluvial channels, the channel limits tend to be at the same elevation since the water that the vegetation (with resulting roughness) uses to grow is also at the same elevation across the watercourse. This requires a field visit.

A secondary consideration is to locate the bank limits where there are obvious breaks in the geometry (e.g., change in bank slope). The location of the bank stations can have an impact on the development of a floodway, as explained later in this document.

Some programs (e.g., HEC-RAS and HEC-2) allow the user to specify breaks in roughness values at locations other than the bank stations. If significant areas have differing roughness (say over 10% of the flow area), subdivide the overbanks and vary "n" by distance across the cross section. Doing this is particularly important if more than one discharge is to be simulated and the roughness changes as the floodplain increases in elevation going away from the channel.

One of the main reasons for subdivision is to separate areas of the cross section where flow is more or less uniform (e.g., main channel flow versus floodplain flow). One recommendation by the U.S. Geological Survey (Davidian, 1984; Thomsen and Hjalmarson, 1991) is to subdivide a cross section with water in both the main channel and the floodplain when the depth of water in the floodplain is greater than half of the maximum depth measured in the main channel.

In any case, the criteria used for subdivision of cross sections should be consistent for the entire reach being modeled. Abrupt changes in channel size and/or shape from one cross section to the next will often result in warnings from the model (e.g., the common HEC-RAS warning "The conveyance ratio (upstream conveyance divided by downstream conveyance) is less than 0.7 or greater than 1.4").

3.3.4 Reach Lengths

For any subsection, such as the channel or overbanks, the representative reach length between adjacent cross sections should be along the flow line that intersects the center of mass of the water in the subsection of the two cross sections. For in-bank flows, the channel distance will most likely be that measured along the thalweg. Reach lengths for both channel and overbanks are often shorter for larger flow events as the water tends to take a straighter flow path.

If necessary, do not hesitate to change reach lengths for different discharges. This can be done in HEC-RAS by creating a Plan with certain reach lengths for a low flow range and another Plan with adjusted reach lengths for a high flow range.

3.3.5 Encroachments, Blocked and Ineffective Flow Areas

One of the most important elements to consider in 1-D modeling is identification of ineffective flow areas. These are areas of the channel or floodplain that may be inundated during a flood event but do not convey water in the downstream direction.

One example of an ineffective flow area is the zone next to an embankment at a bridge crossing. Although the water through the bridge opening is being conveyed downstream, the water immediately upstream or downstream of the bridge approach road is often either stagnant or recirculating in an eddy. Inspection of the flowlines will help in determining ineffective flow areas. In general, if the closest flowline to the boundary starts to significantly depart from the actual boundary, the area between that flowline and the boundary may be ineffective. Figure 3.3 shows an example of how to locate cross sections and use blocked and ineffective flow areas using HEC-RAS for structures in the floodplain. The "rule of thumb" contraction and expansion ratios have been used in this figure. Computed ratios based upon a U.S. Army Corps of Engineers study of flow near bridges (USACE, 1985, 2001) could also have been used.

3.3.5.1 Levees

The modeler will need to check if natural or artificial levees are truly tied to high ground. This is important because it helps to determine the proper modeling technique. Some programs, such as HEC-RAS, have special routines to model levees. These routines will prevent water from entering the area behind a levee as long as the computed water surface remains lower than the defined levee elevation. Only if the levee is overtopped will the model be allowed to include the flow area behind the levee. If the model is part of a Flood Insurance Study, the modeler should look into the specific requirements that FEMA has for levee analyses as they relate to the National Flood Insurance Program.

If there are instances of levee overtopping, the modeler must check for consistency of flow conditions in the overtopped region. For instance, if the model shows that the upstream overbank area has flow in it because the levee was overtopped, the same overbank area of the cross section immediately downstream should also be flowing. If there is a ridge or road on that overbank between the cross sections that forces the flow back into the stream, that feature should have been modeled using additional cross sections.

3.3.5.2 Structures

Structures that affect the flow and water surface profile include bridges, culverts, weirs, and others. It is extremely important to correctly place cross sections and model ineffective flow areas in the vicinity of structures. Modeling of structures is described in Chapter 6 of this Standard.

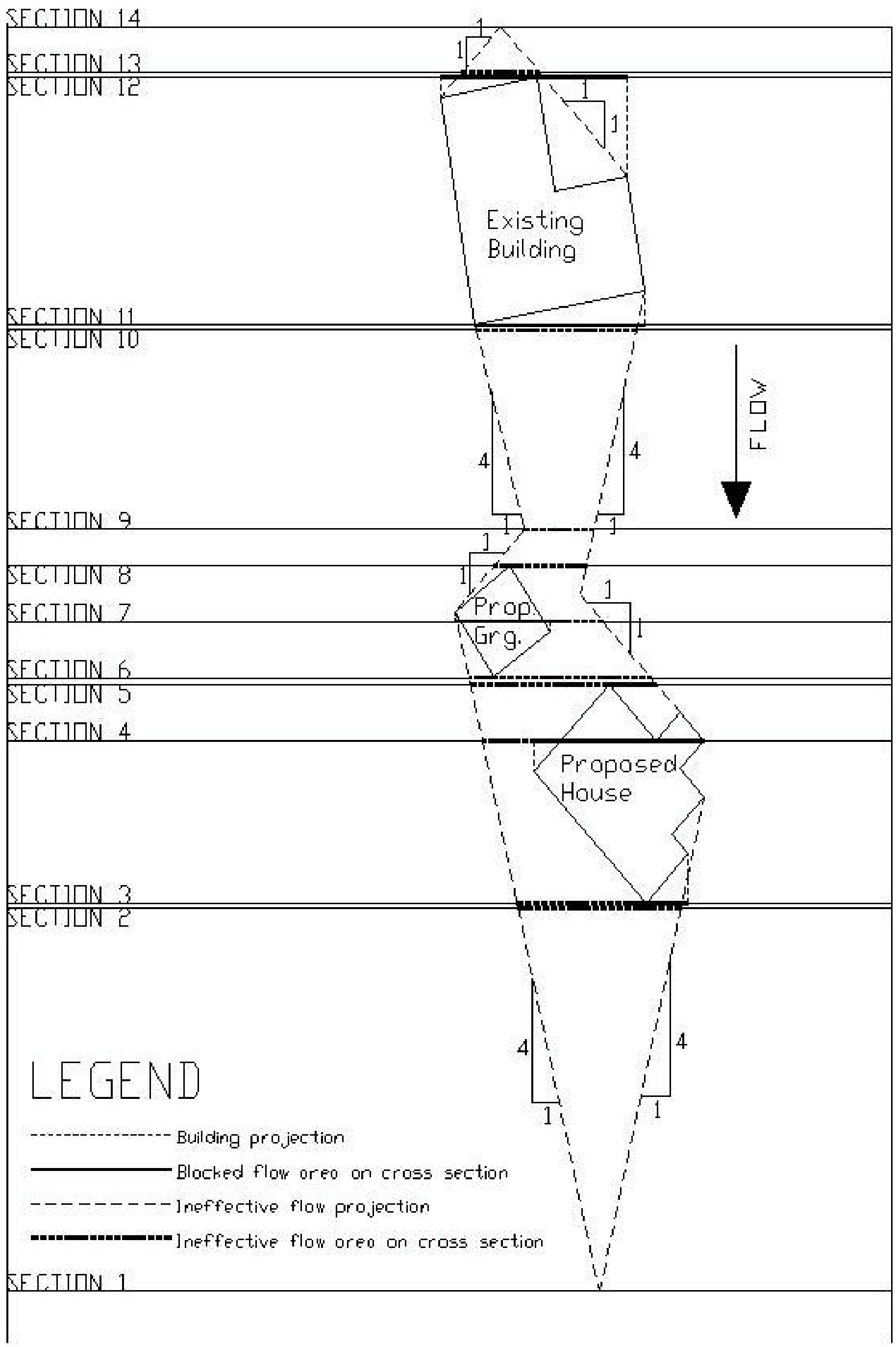
SECTION 14
SECTION 13
SECTION 12
Existing
Building
SECTION 11
SECTION 10
FLOW
SECTION 9
SECTION 8
SECTION 7
Prop.
Grg.
SECTION 6
SECTION 5
SECTION 4
Proposed
House
SECTION 3
SECTION 2
LEGEND
Building projection
Blocked flow area on cross section
Ineffective flow projection
Ineffective flow area on cross section
SECTION 1

Figure 3.3. Modeling Structures on a Floodplain

3.3.6 Interpolation of Cross Sections

Many 1-D modeling programs possess the ability to add interpolated cross sections to the model geometry. This feature can often aid in the convergence to a solution. However, this capability needs to be used with caution. If the modeler finds that additional cross sections are necessary to aid in convergence, the best solution is to obtain these cross sections from survey information, using the interpolated cross section result as a guide to the location and number of cross sections needed.

In cases where the additional data is not available or where the accuracy of the data may not justify obtaining new cross sections, the interpolation feature may be useful. Also, the interpolation feature may be used as a "first cut" to see if adding additional cross sections will result in better model performance. If this is found to be the case, additional cross sections based on survey data should then be added for the final design.

3.3.7 Cross Section Location at Tributaries

Treatment of cross sections at tributary locations will depend on whether or not the tributary is being modeled. If the tributary is not being modeled, cross sections should still be located near the confluence such that water surface elevations are computed at that point. For floodplain studies, it is customary to extend the main stem water surface at the confluence up the tributary using a level pool assumption. Care must be given to accurately capture the effective flow areas on the main watercourse at and near the confluence. Also, cross sections should not be arbitrarily oriented in order to contain flows within the channel geometry, as this violates the one-dimensional model energy assumption.

If the tributary (or distributary) watercourse is being modeled, then cross sections should be placed as near as possible to the junction, but in such a way that the cross section lines do not cross (Figure 3.4). The reach lengths between the downstream-most cross section of the upper reach and the upstream-most cross section of the downstream reach must be along the path of a flow line between the cross sections. That flow line should be the one that intersects the water's center of mass of the adjacent cross sections.

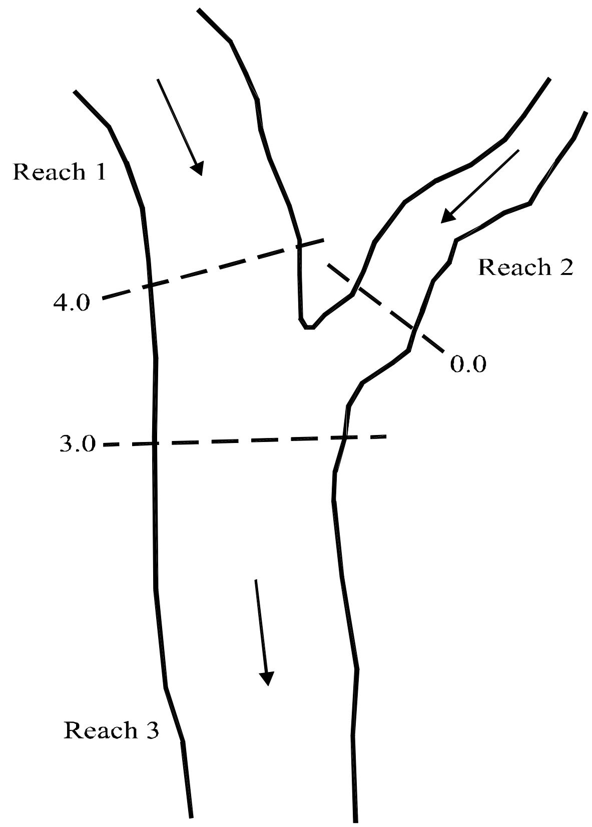

Figure 3.4. Cross Section Placement at Watercourse Junctions

4. Inflows and Outflows

4.1 Local Inflows and Outflows

Most models have the ability to simulate tributary or distributary watercourses, as well as local inflows and outflows. Local inflows and outflows occur at points in a 1-D model where discharge changes due to flow entering or leaving the system and the tributary or distributary watercourses are not modeled. Local inflows and outflows are handled easily in the model by simply changing the discharge at a point or points in the watercourse reach. The change is usually determined external to the program.

4.2 Tributaries, Distributaries, and Breakouts

Modeling of tributaries is usually straightforward since the discharges from each branch upstream of a junction are combined to define the flow in the reach downstream of the junction. Modeling of distributary flow (Figure 4.1) and breakouts (flow leaving the channel along an ill-defined watercourse) is more complex.

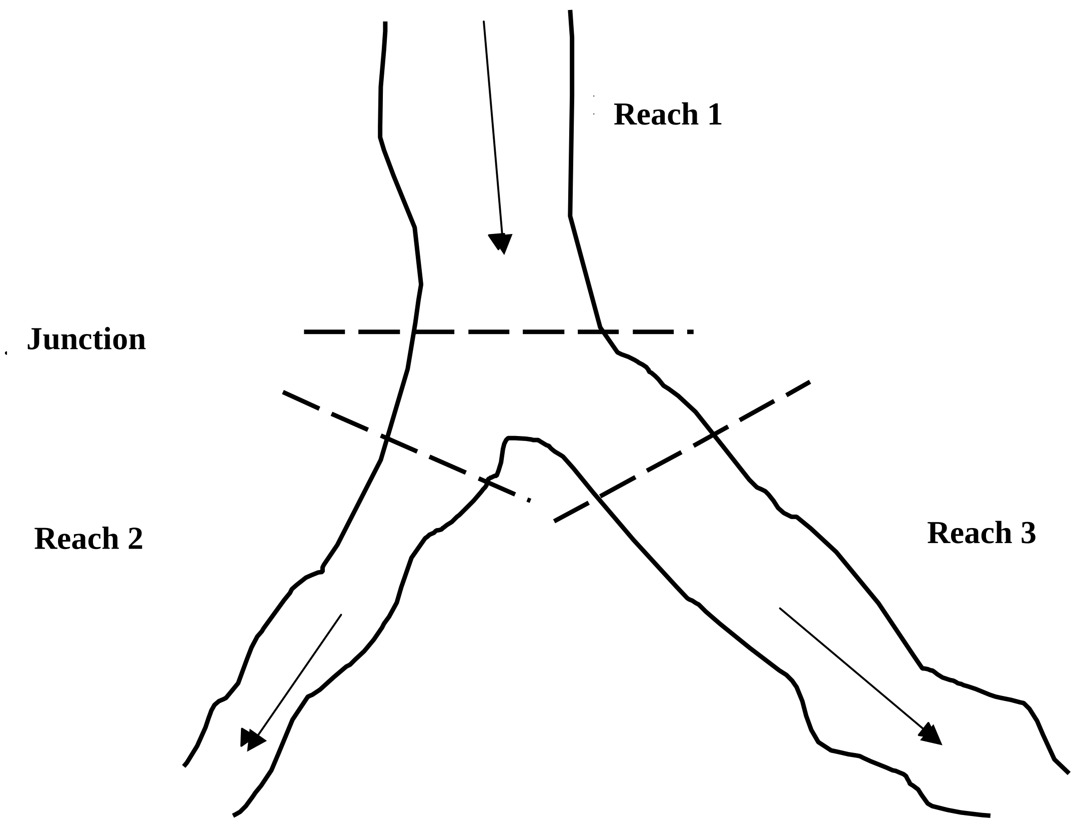

Figure 4.1. Flow Split at a Junction

Some computer programs require the modeler to explicitly define the amount of flow leaving the main watercourse via some external analysis. Other programs (such as HEC-RAS 3.0) will perform iterative calculations using the energy and/or momentum equations to determine how much flow will leave the channel. In HEC-RAS, flow may leave the main channel in one of two ways: either by a flow split at a user-defined junction, or by a user-defined lateral weir along a watercourse reach. The latter method mirrors the "split flow method" found in HEC-2. Water leaving the system via a lateral weir can be brought back (all or a portion of the total) to a downstream location within the model or completely removed from the model.

One simple case of flow leaving the main watercourse and then returning as a tributary somewhere downstream is the situation of flow around an island (Figure 4.2). With 1-D modeling assumptions, the energy at the cross section immediately downstream of the island (River Mile (RM) 10.0 in the figure) will have a unique value, as will the energy at the cross section immediately upstream of the island (RM 10.8). Calculations to solve for the amount of flow in each reach around the island consist of "balancing" the energy at the upstream end of each of the flow paths by an iterative procedure. First a flow split is assumed, and then the energy at the upstream tip of the island is computed for each of the flow paths. The proper division of the total flow in each of the reaches is obtained when the energy at the upstream cross section is the same (within a given tolerance) for each reach.

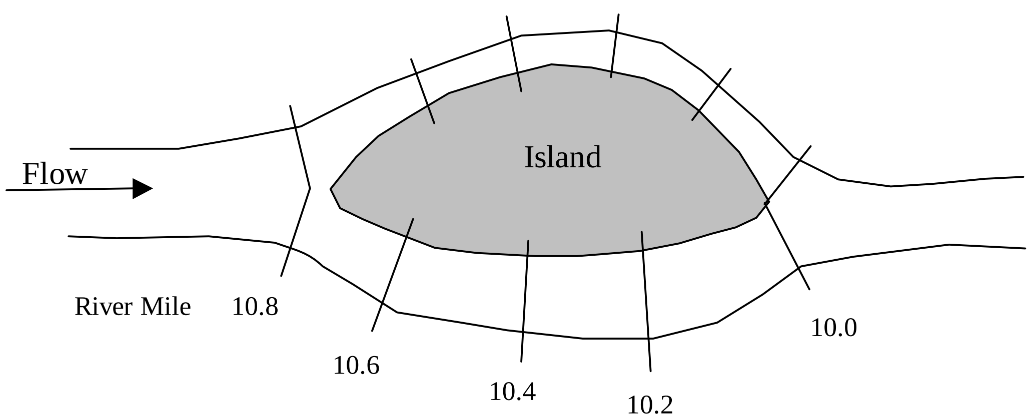

Figure 4.2. Flow Around an Island (Plan View)

For FEMA studies of tributaries, the downstream starting water surface elevation can be based upon either normal depth of the tributary or the main stem discharge when the 100-year flood occurs in the tributary. For that discharge, the water surface elevation of the main stem at the tributary location would then be used as the starting water surface elevation for the tributary. Analyses of the hydrographs (and timing) of the main stem and the tributary must be conducted. The most conservative assumption is usually to use the 100-year water surface elevation of the main stem as the boundary condition for the tributary (coincidental peaks assumption). FEMA

37 (FEMA 1995) states that the assumption of coincident peaks may be appropriate if “a) the ratio of the drainage areas lies between 0.6 and 1.4, b) the times of peak flows are similar for the two combining watersheds, and c) the likelihood of both watersheds being covered by the storm being modeled are high.”

For determination of the FEMA regulatory floodway, the downstream starting water surface elevation is usually based upon normal depth.

5. Special Topics

5.1 Modeling of Hydraulic Structures

Hydraulic structures cause additional energy losses that should be accounted for in the water surface profile calculations with proper modeling procedures. The information and guidelines presented are specifically applicable to HEC-RAS and HEC-2; however, the basic concepts and "rules of thumb" are applicable to any 1-D model. The modeler should always refer to the appropriate computer program user's manual for detailed and specific guidance regarding that particular program.

5.1.1 Bridges

The goal of bridge modeling should be to properly account for the energy losses occurring immediately upstream of the bridge (due to flow contraction), in the bridge structure itself, and immediately downstream of the bridge (due to flow expansion). It should be noted that, for a bridge (and elevated approach roads) that encroach over half of the 100-year floodplain, the hydraulic losses at and near a bridge are due primarily to expansion losses with the contraction loss about the same as the loss at the bridge if there are a modest number of piers. This means that the majority of the modeling effort should be expended in correctly locating cross sections that properly capture the expansion and contraction losses.

The modeler should first consider the kind of flow situation being modeled. There are essentially two types of bridge flow situations: Low Flow and High Flow. The bridge flow computation method should be selected based on the type of flow.

Low flow exists when the flow through the bridge opening is open channel flow and the water surface is below the highest point of the low chord (often referred to as the low steel) of the bridge opening.

High flow exists when the flow through the bridge opening is in contact with the maximum low chord of the bridge deck. When this occurs, the bridge is in pressure flow. If the bridge structure is overtopped, the bridge is under combined pressure and weir flow. If the approach roadway embankments are overtopped, it is usually considered weir flow.

5.1.1.1 General Guidelines

The basic requirements for bridge modeling include:

? **Bridge Cross Sections:** A bridge should be modeled with at least four user-specified cross sections. See "Locating Bridge Cross sections" (Section 5.1.1.2.1) for detailed guidance.

? **Bridge Geometry:** The bridge geometry, both the upstream and downstream faces, should consist of the bridge deck and roadway, abutments, and piers, as appropriate. For high flows that are expected to overtop the bridge, the bridge railing and the approach roadway embankment should be modeled as a part of the bridge deck.

? **Ineffective Flow Areas:** Ineffective flow areas upstream and downstream of the bridge should be based on the type of flow situation being modeled. See “Locating Bridge Cross sections” for detailed guidance.

? **Contraction and Expansion Losses:** Flow contracts as it enters a bridge opening and expands on its way out. Appropriate contraction and expansion coefficients should be used to model energy losses associated with the flow transition through a bridge. The loss due to expansion of flow is typically larger than the loss due to contraction of flow. Abrupt transitions result in larger losses compared to gradual transitions.

? **Selection of Modeling Approach:** The modeler must consider the choice of bridge modeling method based on the type of flow (low flow or high flow) situation. The most basic solution methods available include the energy-based method, the momentum based method, and empirical methods.

5.1.1.2 Specific Guidelines

The following specific guidelines should be followed for modeling bridges.

5.1.1.2.1 Locating Bridge Cross Sections

A plan view of the basic cross section layout is shown in Figure 5.1. The arrows show the actual flow path and the shaded areas shows the ineffective flow areas for non-overtopping conditions.

Cross Section 1 should be located sufficiently downstream from the structure so that the flow is not affected by the structure (i.e., the flow has fully expanded). This distance (also called the expansion reach length) should generally be determined by field investigation during significant flow conditions.

If a field investigation is not possible under these conditions, the study entitled “Flow Transitions in Bridge Backwater Analysis” (USACE, 1995) may be used. Use of the method recommended in the report requires an iterative process and a suggested starting expansion ratio of 3:1 (longitudinal to lateral distance or three times the average of the distance from A to B and C to D in Figure 5.1). Field studies (USACE, 1995) found expansion ratios varying from 0.5 to 4 longitudinal units for each lateral unit.

The expansion distance varies depending upon the degree of constriction, the shape of the constriction and the magnitude and velocity of the flow. If the expansion reach requires a long distance, then intermediate cross sections should be placed within the expansion reach in order to adequately model friction losses.

For non-overtopping flow situations, the ineffective flow area of the intermediate cross sections (between cross sections 1 and 2), as depicted by the shaded areas in Figure 5.1, should be modeled appropriately.

Cross Section 2 is located a short distance downstream from the bridge, commonly placed at the downstream toe of the road embankment. This cross section should represent the effective flow area just downstream of the bridge.

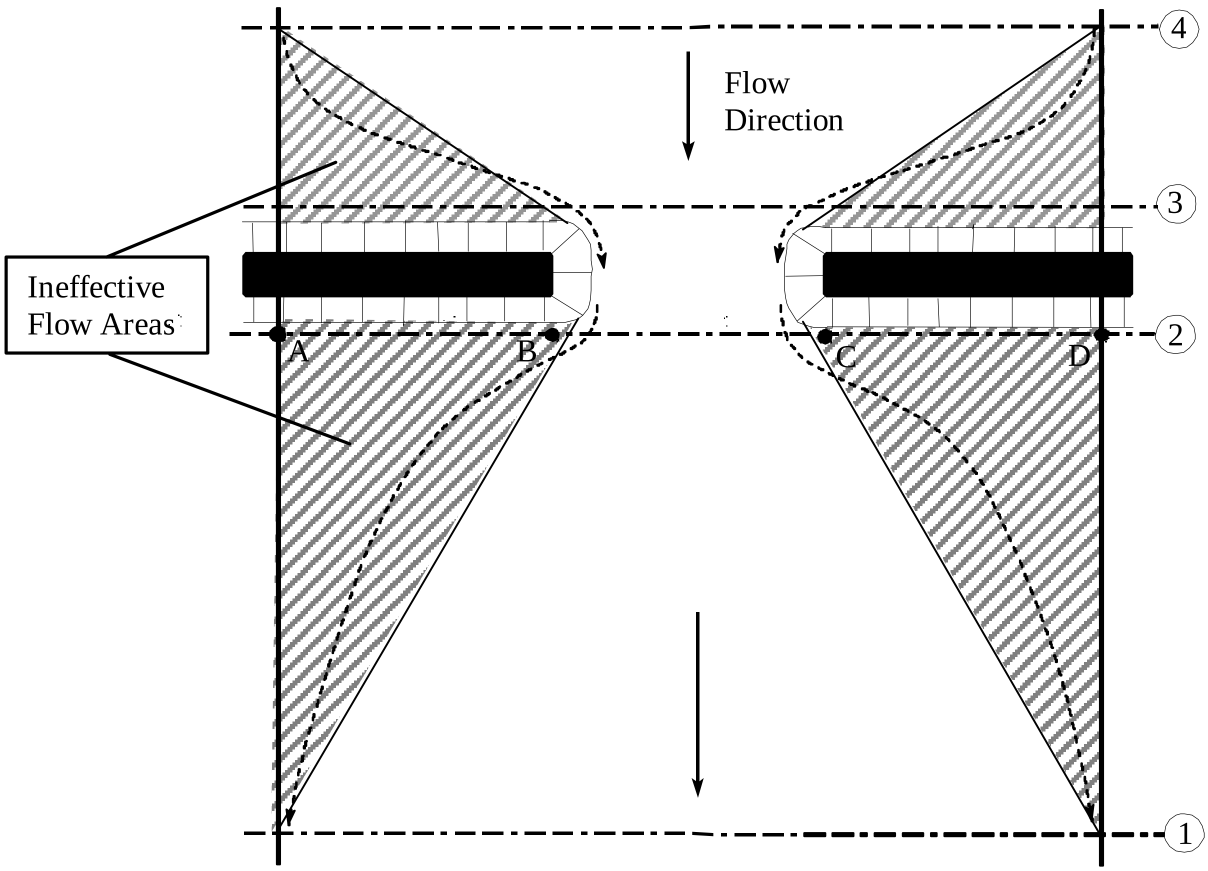

Figure 5.1. Basic Cross Section Layout of Bridge Cross Sections (after USACE, 2001)

For low flow and pressure flow conditions, when the bridge abutments and roadway embankments obstruct the entire floodplain, the overbank areas should be modeled as ineffective (represented by the shaded area between cross sections 1 and 2). For high flows overtopping the bridge deck, the area outside the main bridge opening may become effective flow and should be included as an active flow area.

Cross Section 3 should be located a short distance upstream from the bridge, commonly placed at the upstream toe of the road embankment. The distance between cross section 3 and the bridge should only reflect the length required for the abrupt acceleration and contraction of the flow that occurs in the immediate area of the opening. Cross section 3 represents the effective flow area just upstream of the bridge. Similar to cross section 2, for low flow and pressure flow conditions and when the bridge abutments and roadway embankments obstruct the entire floodplain, the overbank areas should be modeled as ineffective (represented by the shaded area between cross sections 3 and 4). For high flows overtopping the bridge deck, the area outside the main bridge opening may become effective flow and should be included as an active flow area.

Cross Section 4 should be located where the flowlines are approximately parallel and the cross section is fully effective. Because flow contractions can occur over a shorter distance than flow expansions, a suggested starting contraction ratio, generally adopted by the Corps of Engineers, is 1 longitudinal to 1 lateral distance (one times the average of the distance from A to B and C to D in Figure 5.1).

In addition, the study entitled "Flow Transitions in Bridge Backwater Analysis" (USACE, 1995) may be used. Use of the method recommended in the report requires an iteration process and again, a suggested starting contraction ratio is 1 longitudinal to 1 lateral distance (one times the average length of the side constriction caused by the structure abutments). Field studies (USACE, 1995) found contraction ratios varying from 0.3 to 2.5 longitudinal units for each lateral unit.

5.1.1.2.2 Modeling Low Flow

The following guidelines should be followed for selecting the modeling approach for low flow situations:

1) In cases where the bridge piers are a small obstruction to the flow, and friction losses are predominant (e.g., a very wide opening or very few piers), the energy based method or the momentum method may be used.

2) In cases where pier losses and friction losses are both predominant, the momentum method should be used, but other methods may also be applicable.

3) When flow passes through critical depth within the vicinity of the bridge, both the momentum and energy methods may be used.

4) For supercritical flow, both the energy and the momentum method may be used. The momentum-based method may be better at locations that have a substantial amount of pier impact and drag losses.

In HEC-RAS, the user may select any one of the available loss methods, or select multiple methods and have the program use the result with the highest energy loss.

5.1.1.2.3 Modeling High Flow

In general, for high flows the energy based method is applicable to the widest range of problems. The following guidelines should be followed for selecting the modeling approach for high flow situations:

1) When the bridge deck is a small obstruction to the flow, and the bridge opening is not acting like a pressurized orifice (e.g., the bridge is highly submerged), the energy-based method should be used.

2) When the bridge deck is a large obstruction to the flow and a backwater is created due to the bridge deck, the pressure and weir method should be used.

3) When the bridge is overtopped and the water going over top of the bridge is <u>not</u> highly submerged by the downstream tailwater, the pressure and weir method should be used.

4) When the bridge is highly submerged, and flow over the road is not acting like weir flow, the energy-based method should be used.

5.1.1.2.4 Perched Bridges

A perched bridge is one for which the road approaching the bridge is at the floodplain ground level and only in the immediate area of the bridge does the road rise above ground level to span the watercourse. A typical flood-flow situation with this type of bridge is low flow under the bridge and overbank flow around the bridge.

The assumption of weir flow is not justified for this situation because the road approaching the bridge is usually not much higher than the surrounding ground. A solution based on the energy method (standard step calculations) would be better than a solution based on weir flow when a large percentage of the total discharge is in the overbank areas.

5.1.1.2.5 Low Water Bridges

A low water bridge is designed to carry only low flows under the bridge and flood flows are carried over the bridge and road.

When modeling this bridge for flood flows, the anticipated solution is a combination of pressure and weir flow; however, if the tailwater is going to be high, it may be better to use the energy-based method. HEC-RAS automatically uses the energy-based method when submergence is 95 percent or higher.

5.1.1.2.6 Skewed Bridges

Skewed bridge crossings are generally handled by making adjustments to the bridge dimensions to define an equivalent cross section perpendicular to the flowlines.

Skewed crossings with angles up to 20 degrees do not show objectionable flow patterns (Hydraulics of Bridge Waterways", Bradley, 1978). Bridges with a skew angle greater than 30 degrees should be modeled by adjusting the bridge dimensions to represent the projected length of the bridge deck, and if necessary, the projected width of the bridge piers, perpendicular to the flow direction.

5.1.1.2.7 Parallel Bridges

Parallel bridges are usually caused by construction of divided highways or highway and railroad bridges side by side. The hydraulic losses through the two structures may be between one and two times the loss for one bridge (Bradley, 1978).

If the parallel bridges are very close to each other, and the flow will not be able to expand between the bridges, they should be modeled as a single bridge.

If there is enough distance between the bridges in which the flow has room to fully expand and contract, the bridges should be modeled as two separate bridges.

If the bridges are so close that full expansion and/or contraction is inhibited, care should be exercised in depicting the expansion and contraction of flow between the bridges.

5.1.1.2.8 Multiple Bridge Openings

Bridges with side relief openings and separate bridges over a divided channel, with possible different control elevations, are examples of multiple opening problems. The hydraulic analysis of multiple openings is a complex hydraulic problem.

Computer programs such as HEC-RAS have the capability to model multiple openings. An alternative approach is to model the flow paths of each opening as a separate reach.

5.1.1.2.9 Modeling Floating Debris on Piers

Field reconnaissance should be performed to determine the potential for trash, trees, and other debris that may accumulate on the upstream side of a pier during high flow events. Debris may block a significant portion of the bridge opening and should be modeled by increasing the thickness of the piers. HEC-RAS has an option to model floating debris on piers.

5.1.2 Culverts

Culvert modeling is similar to bridge modeling except that culvert hydraulics equations are used to compute inlet control losses. The Federal Highway Administration's (FHWA, 1985) standard equations for culvert hydraulics are the most commonly used. Figure 5.2 illustrates the cross sections required for a culvert model.

Most hydraulic computer programs capable of modeling culverts use the concepts of inlet control and outlet control. The most common culvert types are circular pipe culverts and box culverts. A typical box culvert road crossing is similar to a bridge in many ways with the walls and roof of the culvert corresponding to the abutments and low chord of the bridge, respectively.

The layout of cross sections, the use of the ineffective areas, the selection of loss coefficients, and most other aspects of bridge analysis are applicable to culverts as well. In addition, culvert equations require entrance and exit loss coefficients.

5.1.3 Weirs

Inline hydraulic structures such as weirs, gated spillways, and drop structures (structures across the main watercourse), are typically modeled with empirically based equations such as the weir equation. Computer programs such as HEC-RAS are capable of modeling such structures.

A minimum of four cross sections are required. For drop structures, the cross sections should be closely spaced where the water surface and velocity is changing rapidly. If a sloping drop is to be modeled, additional cross sections should be placed along the slope in order to model the transition from super-critical to sub-critical flow. Several cross sections should also be placed in the stilling basin and the energy dissipation area in order to correctly locate where the hydraulic jump will occur. Manning’s roughness values should be increased inside the stilling basin to account for the increased roughness due to energy dissipater blocks. Note: The design of a drop structure and stilling basin should be performed using empirical equations and methods. The model should provide only the tailwater and headwater elevations and their velocities as input to these equations.

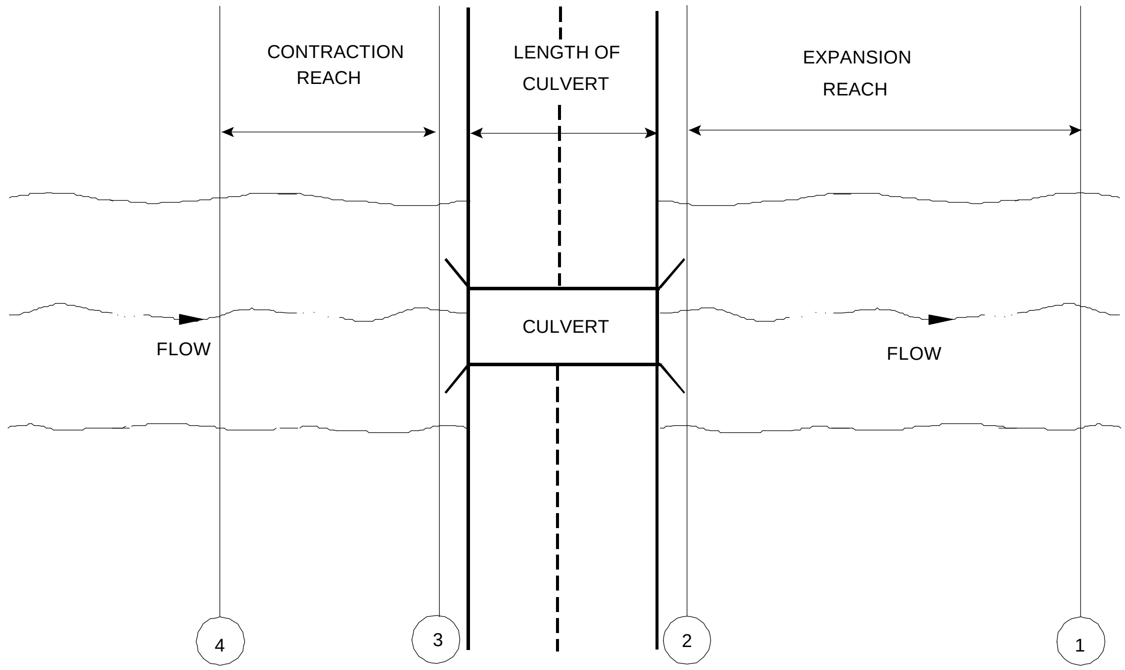

Figure 5.2. Typical Culvert Road Crossing (USACE, 2001)

5.2 Structures in the Floodplain

In urbanized areas, there are often significant portions of a floodplain containing numerous or large buildings that can affect the conveyance of the floodplain. If the predominant flow impedance is due to only a few large buildings, they should be modeled as shown in Figure 3.3. For numerous small buildings that allow flow between the buildings, it may be more appropriate to increase the Manning's "n" value to simulate the hydraulic losses. The choice of how to model a particular situation may depend on whether local velocity and depth information are needed (e.g., for scour calculations), or only water surface elevations are needed. Also, be aware that sturdy fences or walls may sometimes connect these small buildings, forming a continuous barrier to water. If this is the case, it is best to model the continuous barriers as large buildings.

5.3 Selection of Roughness Values

Hydraulic roughness is a major source of uncertainty in water surface profile calculations. Field observations should be supplemented with handbook or analytical methods to predict the roughness values. Resistance to flow depends on a number of factors such as bed material size, changes in channel geometry along a reach, bed forms (dunes, ripples, and other bed forms relating to flow regime), and vegetation type and density. Roughness will also vary with depth, decreasing as the water depth becomes much greater than roughness elements (vegetation, bed material, bed forms, etc.) Therefore, the resistance to flow will vary from season to season and year to year. Because changes will occur over time, a range of roughness values should be considered and a sensitivity analysis is recommended to show how the uncertainty in the roughness value affects the computed water surface elevation and/or velocity.

Consideration should also be given to the overall goal of the model. When velocity is a critical parameter (e.g., in bank protection design), a roughness value on the lower end of the range should be used, and when the water surface elevation is more critical (e.g., in levee design), a higher roughness value should be used.

Handbook methods involve the use of "calibrated photographs" and other subjective methods to associate hydraulic roughness values with conditions observed and anticipated in the project reach. Chow (1959), Barnes (1967), and more recently Hicks and Mason (1998) are the dominant sources of calibrated photographs. Arcement and Schneider (1989) published a report with photographs to help estimate roughness values for vegetated floodplains.

Analytical methods are physically based equations that relate hydraulic roughness to the effective surface roughness and irregularity of the surface boundaries (Example: Moody Diagram). Notable equations that can be used are the Strickler equation, Keulegan's rigid bed equations, the Iwagaki relationship, the Limerinos equation, and others (e.g., USACE, 1994).

For roughness value estimation, the following publications specific to Arizona channels may be consulted:

1) Estimated Manning's Roughness Coefficients for Stream Channels and Floodplains in Maricopa County, Arizona, (Thomsen and Hjalmarson, 1991)
2) Roughness Coefficients for Stream Channels in Arizona (Albridge and Garrett, 1973)
3) Verification of Roughness Coefficients for Selected Natural and Constructed Stream Channels in Arizona (Phillips and Ingersoll, 1998)

For the design of vegetated flood control channels, the modeler must use an appropriate roughness value to account for fully vegetated conditions. One reference specific to Arizona watercourses is "Method to Estimate Effects of Flow-Induced Vegetation Changes on Channel Conveyances of Streams in Central Arizona" (Phillips et al., 1998).

A word of caution in using Manning's "n" values based upon a field reconnaissance. When the modeler inspects a watercourse to judge its roughness, the physical state of the system when it is inspected is not necessarily the physical state the system would be under design conditions. FEMA studies are usually based upon a 100-year flood, and the roughness of the watercourse during such an event would be drastically different than what is seen in the field. What is seen in the field may be several years after a major flood event and it may have "healed" or been worked over by smaller floods, hiding any evidence of how the actual situation was during high flow events. It is suggested that if the roughness is based upon vegetative resistance, a shear stress analysis should be conducted to check if the vegetation's critical shear stress is exceeded by the actual shear stress. Conversely, if a plane streambed is observed and only the grain resistance is used to obtain the "n" value, the actual resistance may be higher under design floods because dunes or ripples may form and become important additional components in determining the Manning's "n."

5.4 Bends

Cowan (1956) has suggested that the Manning's "n" value can be adjusted to simulate the additional hydraulic loss due to bends in the watercourse. He suggests modification to the overall n value based upon the severity of the bend. However, an adjustment is generally not needed for a watercourse with a ratio of the radius of curvature to active channel width greater than 10. Also, some methods used for determining the roughness value (e.g., "calibrated photographs") implicitly include energy losses due to bends. Changing expansion and contraction coefficients to simulate bend losses is not only inappropriate, there are no guidelines on what are reasonable values to use for various degrees of bend severity. Adjustments to the floodplain due to superelevation at bends are not suggested in FEMA studies; however, they are commonly used for designing flood control protection such as artificial channels and levees and for determining increased shear stresses for streambank protection measures.

5.5 Modeling Supercritical and Mixed Flow

All 1-D models will compute solutions for subcritical flow conditions. Many will also compute supercritical flow. A few, such as the steady flow module of HEC-RAS, will compute "mixed flow," i.e., both sub- and supercritical flow occurring in a single water surface profile. The Arizona State Standard on Supercritical Flow (SS3-94) should be referred to when modeling supercritical flow situations. Note: For FEMA floodplain studies, supercritical flow results from hydraulic models will not be accepted by FEMA for delineation of floodplains or floodways unless the watercourse was specifically designed for and adequately protected against supercritical flow conditions under the 100-year flood discharge. Also, if the watercourse was designed to accommodate supercritical flow, the floodplain becomes the floodway since the concept of a regulatory floodway based upon an "allowable rise" is not appropriate for supercritical situations. This point is also emphasized in SS3-94.

5.6 Floodplain Delineation Models

Delineation of floodplains for a Flood Insurance Study (FIS), letter of map revision, or other purpose is a common reason for developing/updating a hydraulic model of a watercourse. FIS requirements are provided in the FEMA 37 document (FEMA, 1995) and other State Standards. New FIS's and revisions to existing studies should follow the guidelines in these publications and the modeling procedure given in Section 6.3 of this standard.

5.7 Modeling of Pit Areas (sand & gravel mining) and Lakes

Sand and gravel extraction operations result in pits or depressions that can be located in-channel or off-channel. In-channel pits are less common because of the environmental impacts, the related regulations, and permitting procedures. Off-channel pits are more common and are located in the overbank area adjacent to the main channel. Off-channel pits are typically separated from the main channel by levees. Similarly, natural or man-made lakes or depressions can be found either in-channel or off-channel. There is essentially no difference between lakes and pits with regard to hydraulic computer modeling for floodplain studies.

5.7.1 In-Channel Pits

With regard to a hydraulic study involving an in-channel pit, one of the important factors influencing the type of model selected is the size of the pit. If the pit is very large in comparison to the average channel size (such as a lake), storage and its effect on flow attenuation will be a major consideration. In that case, an unsteady flow analysis (which considers the finite volume of water in the hydrograph) may be the best choice.

On the other hand, the purpose of the study may govern the kind of model selected. If the purpose is to delineate floodplains, it might be of interest to compute the most conservative (maximum) water surface elevations. In such a case, a steady flow model can be selected and the water surface elevation computed by completely ignoring the pit (filling in the pit) in the cross section.

For example, the steady flow module of HEC-RAS can be used to model the pit as a blocked obstruction, flush with the bed surface. Such a method would eliminate the additional conveyance in the pit and result in higher water surface elevations. Blocking conveyance in the pit area by using high Manning's "n" values is not recommended because an unrealistically high roughness will be used for a whole range of flows (unless the model has the ability to vary the roughness with stage or discharge). In general, if conservative results are desired, in-channel and off-channel pits should be modeled as blocked obstructions flush with the adjacent streambed. The "n" values used for the pit area should be consistent with the roughness values assigned immediately upstream and downstream of the pit area.

Other factors to consider are the magnitude of the flood and the potential for sediment transport. If the pit is small, it may be reasonable to assume that the pit may be filled with sediment during a large flood. This assumption would then become the basis for ignoring isolated depressions and smoothing out the cross section for use in a steady flow analysis. On the other hand, if a flood of small magnitude were being modeled, it may be necessary to account for the presence of the pit.

Another factor that could be considered is the shape of the storm hydrograph. The hydrograph should be examined in order to determine if the pit will be filled before the peak of the storm hydrograph arrives at the location of interest. The pit will most likely be filled if the hydrograph is slow to peak and has a rising limb with a volume close to or greater than that of the pit. In this case, it may be justified to assume that the pit can be blocked off and does not possess any conveyance during the modeling of the peak discharge in a steady flow analysis.

5.7.2 Off-Channel Pits

For off-channel pits, the major considerations are the presence of levees, the size of the pit, and the magnitude of the flood. If the levees separate the main channel from the pits and are assumed to hold during the flood under consideration, the area beyond the levees can be ignored or blocked off. If the levees are failed or if the flood is large enough to overtop the levees, two situations should be considered.

In one situation, the pit area will be storing water but not actively conveying flow downstream. In this case, the pit should be modeled as an ineffective flow area.

In the second situation, the pit area is actively conveying flow and will have to be modeled as a part of the overbank flow path. If a steady flow model is being used, a split flow analysis should be conducted to determine the breakout discharge from the leveed main channel. Then, based on the overbank flow conditions, the overbank area including the pit can be modeled as a separate reach conveying the breakout flow. If an unsteady flow model is being used, the pit in the overbank area can be modeled as a storage cell defined by an elevation-storage curve and/or as a separate flow path similar to the discussion for the steady flow model.

In summary, a steady flow analysis should be selected by ignoring the pit if conservative results are desired and the pit is small. An unsteady flow analysis should be considered when flood storage is a major consideration such as for large in-channel pits and lakes or when there is the possibility of flow through or over levees into depressed floodplains containing a large pit.

5.7.3 Model Input Data Considerations

Model selection is followed by the collection of model input data. At a minimum, such data includes topographic maps or cross section surveys, as-built plans for bridges, or other in-channel structures. Proper consideration must be given to the selection of the scale and contour interval of the topographic map. For example, if an off-channel pit is being considered as a storage area, elevation-volume data of the pit must be obtained from the topographic map. In this case, it may be necessary to obtain a map with a 1-foot or a 2-feet contour interval. Often, the mapping costs constitute a significant portion of the total cost of the modeling study.

The most important task before data entry to the hydraulic model is the proper orientation and spacing of the cross sections and review of the cross section data. Cross sections should always be aligned perpendicular to the direction of flow for the event(s) being modeled and spaced such that major grade breaks in the channel profile are represented. This is particularly important if the pits or related activities change the natural flow patterns. Section 3.3 provides additional information on cross section placement and orientation.

For modeling large pits and lakes, an appropriate number of cross sections should be selected to properly represent the change in pit dimensions. At a minimum, there should be a cross section located at the beginning and the end of the pit with an additional cross section at the widest part of the pit. The model will then “see” the pit as a diamond shape in planform. If this shape does not adequately describe the actual shape of the pit, then additional internal cross sections may be needed.

6. FLOODWAY METHODS

6.1 General

The Federal Emergency Management Agency (FEMA) administers the National Flood Insurance Program (NFIP). Under this program, a regulatory floodway is defined as the channel of a watercourse and the adjacent land areas that must be reserved in order to discharge the base flood without cumulatively increasing the water surface elevation by more than a designated height. The area between the limits of the regulatory floodway and the inundation limits for the base flood is called the floodway fringe. The base flood is the event having a 1% chance of occurring in any given year (also called the "100-year" flood). The designated height is one foot (the maximum allowed by FEMA) or a lesser amount specified by a local agency. For near-critical or supercritical flows, the height is applied to the rise in the energy grade line rather than the rise in water surface elevation.

The local community is responsible for prohibiting encroachments in the regulatory floodway (including fill, new construction, and other improvements) unless it has been demonstrated through hydrologic and hydraulic analyses, performed in accordance with standard engineering practice, that the proposed encroachment would not result in any increase in flood levels during the occurrence of the base flood discharge. Proposed encroachments in a designated floodway, which result in an increase in the base flood elevations (BFE's), must be approved by FEMA. Such encroachments must comply with §60.3.d.4 and §65.12 of the NFIP regulations. A schematic of floodway definitions is shown in Figure 6.1.

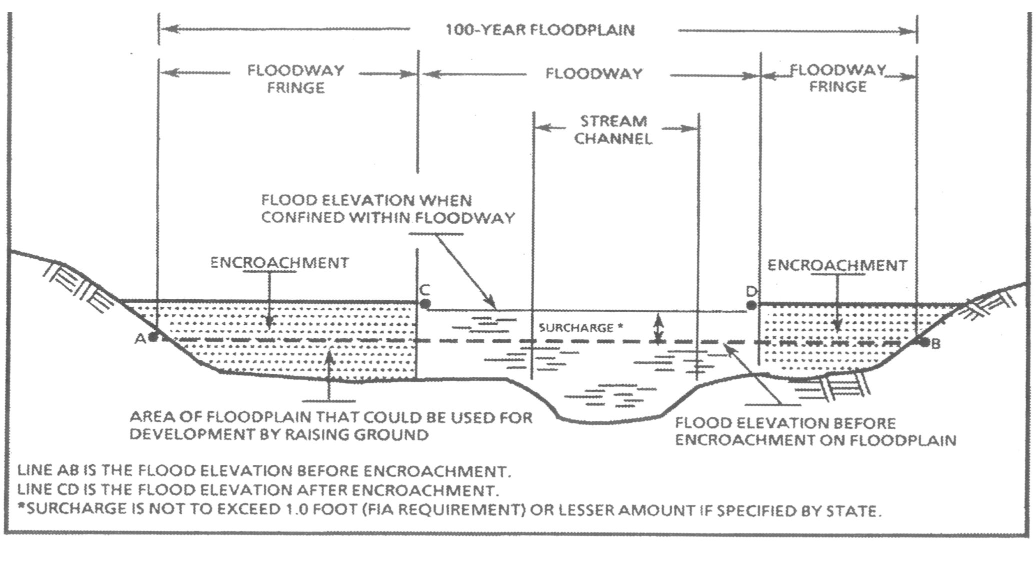

Figure 6.1. Schematic Showing Floodway Definitions

6.2 Floodway Development

This section provides a general procedure to evaluate the impact of a development (hereby referred to as "floodway development" or "floodway encroachment") inside a regulatory floodway limit. Floodway development has the potential to obstruct flow. But since such development, by virtue of being in the regulatory floodway, is subject to the "no-rise" criterion for base flood elevations (explained in greater detail later), it usually requires the implementation of terrain modifications (hereby referred to as "compensatory actions") to insure that flood levels do not rise as a result of the floodway development. There are two possible ways to achieve the compensatory actions: by excavation, or by the reduction of the roughness coefficient. Note: Some local floodplain management ordinances regulate what uses are allowed in a floodway. Prior to modeling a development in a floodway, the modeler must check with the local floodplain management office.

FEMA generally requires that the same software package used for the original Flood Insurance Study also be used to model the modifications to the floodway. However, newer and improved versions of the same software package are often available. FEMA generally permits the use of newer software versions for development studies. However, newer software versions will sometimes compute different water surface elevations than previous versions. For example, Version 3.0 of HEC-RAS will often give slightly different water surface elevations than identical models run under Version 2.2 of HEC-RAS. Therefore, a "duplicate effective" model needs to be created, which is a copy of the original model (the "effective model") with water profiles re-calculated with the newer software version. For uniformity, the "duplicate effective" model should be created whether the software version has changed or not.

6.3 Modeling Procedure

The general procedure recommended for floodway encroachment analysis is a series of 5 different models: Duplicate Effective, Corrected Effective, Pre-Project (Existing) Conditions, Post-Project without Compensatory actions, and Post-Project. Each model is an incremental modification of the model preceding it. Each model also has two different versions: the unencroached version (no floodway) and the encroached version (with floodway). All these models are run using the same version of the software. A conceptual description of each model is provided below.

6.3.1 Duplicate Effective Model

The Duplicate Effective model is a copy of the original FEMA Flood Insurance Study model (called the "effective model"), where the water surface profiles are computed using the version of the software (usually the most recent version) that will be used for the remainder of the floodway development study. This model should have all the original encroachment stations so that the model can be run in the unencroached and in the encroached versions. If the software version for the floodway development study is not identical to that used in the original Flood Insurance Study, the water surface elevations in this model might differ from those in the Effective model. If this is the case, the reasons for the differences should be established. The results from the newer software version are generally accepted by FEMA.

6.3.2 Corrected Effective Model

The Corrected Effective model is the Duplicate Effective model with the following modifications:

1) It contains additional cross sections that will be used in the Pre- and Post-Project models (e.g., if a bridge is proposed, the 6 cross sections needed for bridge modeling should be added). In the corrected effective model, the cross section geometry and "n" values should reflect the *conditions at the time the Flood Insurance Study was conducted.* That means that no changes to the topography since the Flood Insurance Study was conducted are to be considered.

   The encroachment stations (floodway limits) for all the added cross sections must be determined. This is done by plotting the new cross sections on the Flood Insurance Rate map and measuring the floodway limits (which are the encroachment stations) using this map.

2) Any conceptual modeling errors in the original FEMA model can also be corrected in this model with the reasons for the corrections well documented. However, unless the errors are significant, it is suggested that no additional modifications be made.

It is frequently the case that the addition of cross sections in this model will cause changes in the calculated water surface elevations. This brings up two issues:

1) <u>The base flood elevations change; i.e., the water surface elevations in the unencroached model.</u> The addition of cross sections alone will usually cause the model to compute different water surface elevations than those computed in the Duplicate Effective model, in the cross sections upstream of the new cross sections.

2) <u>The encroached, or surcharged, water surface elevations change; i.e., the water surface elevations in the case where the floodway fringe is excluded from the flow area.</u> The criterion that must be met is that the encroached water surface elevations must be one foot or less than the unencroached water surface elevations at all cross sections. If these "surcharges" are greater than 1', then the floodway must be redelineated.

6.3.3 Pre-Project (Existing) Conditions Model

The Corrected Effective model cross sections are modified to reflect any changes in the topography that have occurred since the Flood Insurance Study was conducted. These modifications apply to the cross sections added in the Corrected Effective Model as well as to the original cross sections from the Effective model. Changes that are near the proposed project site need to be incorporated in the Existing Conditions model. Changes at cross sections not near the proposed site typically need not be modified.

Generally, the Existing Conditions model will not differ from the Corrected Effective model unless there has been some natural deposition or erosion. Any man-made changes within the floodway should have been submitted to FEMA for approval, and the Flood Insurance Study model should have been updated to reflect these changes. If there have been undocumented changes inside the floodway since the Flood Insurance Study, this would be a special case requiring consultation with FEMA and the community on how to proceed with the development study.

6.3.4 Post-Project Without Compensatory Actions Model

For the Post-Project without Compensatory actions model, the cross sections are modified to reflect the new structure or project. Compensatory actions are not included in the model. This intermediate model is not required by FEMA, but is recommended for clarity.

6.3.5 Post-Project Conditions Model

For the Post-Project Conditions model, cross sections are modified to reflect the compensatory actions needed to result in a "no-rise" condition, explained in the following section.

6.4 "No Rise" Condition

The "no rise" condition means that the water surface elevations at every cross section in the Post-Project unencroached model cannot exceed the water surface elevations at each corresponding cross section of the Existing Conditions unencroached model. Satisfying this condition demonstrates that the project will not cause a rise in the BFE's.

An additional consideration is the effect of the development on the encroached water surface profile. This could occur, for example, if a building was added in the floodway, and an excavation was conducted in the floodway fringe to compensate. In that case, the BFE calculations (which are unencroached) might result in no increase of the water surface, but the encroached water surface elevations could rise more than one foot because the compensatory action in the fringe would provide no additional conveyance in the regulatory floodway.

Except for encroachments in designated floodways, the NFIP regulations permit a rise in the encroached water surface elevations as long as the surcharge remains less than or equal to one foot. However, the local community may wish to apply the no-rise criteria to the encroached profile for several reasons. For example, any compensatory action taken in the floodway fringe would need to be permanent, causing an enforcement issue to maintain properties outside the floodway. Also, new developments in the floodway fringe could eliminate the benefit provided by a compensatory action, making it difficult to guarantee the permanent hydraulic effectiveness of the compensatory action.

Therefore, it is recommended that all compensatory measures be located inside the floodway, that they be "permanent" (e.g., maintained and hydraulically connected) measures, and that the "no rise" condition be applied to the encroached Post-Project model as well as the encroached Existing Conditions model.

6.5 Cumulative Effects

The cumulative effects of development in the floodplain should not violate the "no rise" condition described in the preceding section. Where no floodway has been designated, conducting a re-study and developing a floodway usually addresses the cumulative effects. Revised discharges and updated topography may be required for the re-study.

6.6 Generation of Additional Cross sections

Generally, numerous cross sections must be added in the Post-Project model (these cross sections appear in the Corrected Effective and subsequent models as well) to properly model the floodway development. For example, in the case of a building added in the floodway, a minimum of six cross sections would need to be added to properly model the building. Sometimes even more cross sections must be inserted for developments with overlapping hydraulic effects (see Figure 3.3).

Such additional cross sections can be surveyed, or they can be developed from topographic maps. Unless the modeling conditions are very simple, however, it is often not easy to determine where cross sections will be needed. Therefore, the recommended procedure is to develop a topographic map of the area if none exists.

Sometimes, in order to create all the cross sections needed for the hydraulic model, cross sections are copied from other cross sections nearby. This practice should be restricted to copying from cross sections that are very close and where there would not be any significant changes in shape or elevation between the two cross sections. Such actions should be documented within the model and report. In general, FEMA does not accept copied cross sections for floodplain and floodway determination.

6.7 Reducing Roughness

Often, an expedient way to lower the calculated water surface elevations is to reduce flow resistance by selective clearing and landscaping, thereby reducing Manning's "n" values in the model at or near the floodway development site. FEMA requires that a certified maintenance plan be submitted (FEMA, 1990) when this method is pursued.

Reduction of roughness can be achieved, for example, through changes in landscaping or through paving. **However, even with certified maintenance plans, reductions in roughness achieved through landscaping must be viewed skeptically**. Due consideration should be given to the long-term costs of monitoring the modifications, the funding mechanism for meeting the costs, and the likelihood of the permanence of the modifications. Modifications not subject to high maintenance, such as paving with concrete, provide more confidence in their durability. Whatever the method used to reduce roughness, such a reduction usually results in increased channel velocities. The effect of the new velocities on channel banks and structures needs to be considered.

Increases in conveyance can also be more reliably and permanently achieved through excavation and grading. The excavation must be of such size and extent that it will not act as merely a depression, but rather as a free-flowing conveyance element during times of flooding. However, in areas of high sediment transport rates, the excavated area may be filled after major storms. Therefore, if this situation is highly probable, periodic surveys may be needed to assure the required conveyance is still available.

6.8 Comparison of Conveyances

A method using the comparison of conveyances to analyze a development inside a floodway is detailed in a FEMA memo from the administrator of the Federal Insurance Administration dated

October 31, 1990. This memo implies that a backwater calculation will nevertheless be required in addition to the conveyance calculations. FEMA requires a backwater calculation using a computer model. Therefore, the comparison of conveyances can only be considered an auxiliary to a backwater calculation, not a substitute.

The inclusion of conveyance comparisons is a courtesy to FEMA. The comparison should be between the conveyance of the encroached Existing Conditions model and that of the encroached Post-Project model. These conveyances can be extracted directly from the respective computer models. Conveyances should be compared in every cross section where there is a geometric and/or roughness difference between the Existing Conditions Model and the Post-Project model.

In a “no-rise” scenario, the conveyance in the Post-Project model will generally not differ from that of the Existing Conditions model, but this is not always the case. The post-project conveyances at some sections could decrease because of compensatory actions achieved at downstream cross sections. Alternately, the compensatory actions might have been taken outside the floodway (a practice already discussed that is not recommended).

Note: A decrease in conveyance would not be grounds by FEMA for the rejection of the proposed development. There might be no increase in water surface elevations from the Effective to the Post-Project Models, but there may be decreases in conveyances at some cross sections. Because a conveyance increase could signal an error in the model, and may be considered a “red-flag,” it is recommended that an explanation of how the “no-rise” condition could be achieved under such circumstances.

7. Good Modeling Practice

Hydraulic models require large amounts of data and can produce misleading results without proper planning, organization, and execution. A well-managed analysis can save time and money. This section emphasizes good modeling practices that can increase the efficiency of the modeling effort and improve the quality of the results.

7.1 Model Documentation

Proper model documentation is perhaps one of the most important and yet one of the most neglected aspects of hydraulic modeling. Experienced modelers understand that good modeling practice also includes proper documentation and file management. A properly documented model can save much time and effort in the long run and helps expedite the review process.

Since hydraulic models are based upon accurate depiction of the watercourse geometry, it is imperative that the horizontal and vertical accuracy of the survey is adequately described. In addition, the vertical datum used should be provided in the report and any rectification of geometric data sources using different datum should be described in detail.

The report should include topographic maps with cross sections overlaid on them, including their orientation. Also, to assist the reviewer, it is suggested that the flowlines and ineffective flow areas in relation to the cross sections be shown on topographic maps.

A modeling project involves the development of computer model input and output files usually copied on a floppy diskette or CD and included with a project report. However, diskettes or CDs frequently get separated from the reports, and without proper documentation the input data files are practically meaningless. Therefore, basic information should always be included in the input data files and cross-referenced with more detailed explanation in the report.

If the modeling effort involves several plans with multiple input data files, each file should contain enough documentation to explain the differences between the input files. It is also useful to name input data files in a creative and unique manner that helps to quickly identify the nature of the plan or model run.

All input data files should contain basic information such as the following: the name and location of the project, the name of the watercourse, the name of the modeler, the company the modeler works for, the address and phone number of the company/modeler, the date of the run, the input file name, and a brief description of the main purpose of the run. Additional documentation such as model source data, explanatory comments on warning messages, and notes on modeling decisions, may also be included. At the conclusion of the modeling exercise and after the numerical output has been checked, the modeling notes in the input data file should be reviewed, and any unnecessary comments that could cause confusion to a reviewer should be removed.

Many modeling programs can accommodate comments by the modeler in the program input and/or output. Frequent comments can greatly aid a reviewer, as well as anyone who uses the model in the future.

Project reports should contain a detailed discussion of the model input and output along with reference to the model input data filenames, where necessary. For projects that involve multiple input data files, the report should include an appendix that contains a list of all the input data files and a brief description of each file. The Arizona State Standard on Flood Study Technical Documentation (SS1-97) should be referenced for more information.

7.2 Common Errors

This section describes the checks for common modeling errors in typical floodplain delineation studies. A review of errors related to special computations such as bridges, split flow optimization, junction analysis, and floodway computations are addressed elsewhere in this attachment.

A hydraulic analysis begins with properly laid out cross sections and estimates of channel roughness and other hydraulic parameters. Tasks such as cross section layout may seem mundane, but should always be supervised by an experienced modeler. This is especially true for floodplains in the arid southwest where the flow paths are sometimes hard to determine. The cross section spacing should be determined based on the change in the slope of the ground profile of the watercourse as well as the change in the width of the watercourse. In general, the cross section spacing is determined by the need to adequately account for the energy losses (friction, flow expansion, and flow contraction) between consecutive cross sections.

Generally, the most obvious errors that appear in the model output should be eliminated first. In a HEC-2 analysis, these errors may show up as messages in the EDIT-2 output or at the end of the HEC-2 output file. In a HEC-RAS analysis, these may show up in the "errors, warnings, and notes" window.

After correcting the obvious errors, a detailed review of the graphical and tabular output (summary tables) should be conducted. The graphical output is especially useful in locating large changes in water surface elevation, top-width, slope, or velocity. Sudden changes in hydraulic parameters should be based on physical evidence and be explainable in a logical manner. Graphical output usually consists of water surface profiles and cross sectional views.

One of the most important variables in the output is the energy slope. If the slope increases or decreases rapidly between consecutive cross sections, it usually indicates the need for decreasing the cross section spacing and adding more cross sections.

If the output indicates critical depth at isolated cross sections, check the geometry of the cross sections. If no coding errors are found, check the change in energy slope between cross sections. A large change in energy slope may indicate the need for additional cross sections or that ineffective flow areas need to be defined. If critical depth occurs at several consecutive cross sections, this may indicate that supercritical flow exists. If using HEC-2, a supercritical analysis may be required, and in HEC-RAS, a mixed flow run may be more appropriate (see modeling guidelines in the State Standard for Supercritical Flow, SS 3-94). However, it should be noted that FEMA does not permit the use of supercritical profiles in alluvial channels. Therefore, the critical depth results may be of less concern for flood insurance studies.

If the output indicates a dramatic change in the top width of the effective flow area between cross sections, the flow paths should be checked on a topographic map. Additional cross sections may be required to better define the flow path in this case. A sudden change in top width may also indicate an error in the coding of levees, not specifying areas of ineffective flow, or blocked obstructions.

If there is a large change in the distribution of flow in the channel and overbanks between consecutive cross sections, this may indicate the need for additional transitional sections.

Areas of ineffective flow or ponded water should not be included in the effective flow area. Such areas may be simulated using the ineffective area option or by using a high roughness value.

Isolated messages indicating divided flow (a condition where wetted areas of the cross section are divided by one or more dry areas for a given water surface elevation) are not of concern unless the cause of the divided flow is continuous for a long distance down the watercourse. If this is the case, there will be several consecutive divided flow messages. The flow path should then be checked on the topographic map to see if there are isolated flow paths that cannot "hydraulically level each other" and that do not have the same water surface elevations along the same cross section. Should this occur, a separate analysis should be performed for each flow path (see the flow around an island discussion in Chapter 4).

The cross section station or river mile designation should be checked to ensure it corresponds to the cumulative channel reach length distance. Overbank distances are usually shorter than the channel distance.

Skewed cross sections should be corrected by using the projected length. The projected length is found by projecting the skewed length onto a plane perpendicular to the direction of flow. In some cross sections, the skew may apply to only a part of the cross section such as the channel, and not the overbanks. In such cases, the cross section length should be adjusted.

Interpolated cross sections should be checked carefully to ensure that they represent the channel and overbank conditions accurately (especially in HEC-2 modeling). The channel flow-line elevation of a copied cross section should be checked to make sure that it represents the actual flow-line elevation at that location.

The output should be scanned for warnings about vertical extensions of the ends of the cross section. These warnings occur when the computed water surface elevation exceeds the ends of the cross section. In this case, the cross sections should be extended based on topographic data.

The selection of the roughness value is the single most important parameter that determines the accuracy of water surface profiles. Roughness values should be estimated by an experienced professional, and the report should address the method of selection and the appropriateness for the purpose of the project. The selected roughness values should be consistent throughout the model. If possible, roughness values should be calibrated to measured water surface elevations.

The selection of the expansion and contraction coefficients can also have a significant influence on the accuracy of the water surface profile calculations. Values should be based on the criteria mentioned in the appropriate user's manuals. Contraction and expansion coefficients should generally be increased at constrictions such as bridges and culverts.

Multiple profile runs should be checked to ensure that there are no conflicts in the modeling requirements for the various profiles. For example, the definition of ineffective flow areas and roughness values selected for modeling a high flow situation may not be applicable for a low flow situation.

Many projects require the use of before and after comparisons such as "existing conditions" and "proposed conditions" simulations. It is important for both simulations to have cross sections at the same locations to ensure a proper comparison of the results.

Model limits should be sufficiently downstream of the project so that computed water surface profiles in the area of concern (project area) are not influenced by downstream boundary conditions. Likewise, the model limits should extend far enough upstream of the project such that any project effects (e.g., backwater) are captured.

7.3 Quality Control Checklist

The following is a minimum checklist for quality assurance and control. The proposed checklist should include the following and any particular guidelines presented in other sections.

The modeler and/or the reviewer many wish to create specific checklists based upon the types of floodplain studies. Appendix C provides an example of a HEC-RAS Reviewer's Checklist.

7.3.1 Check Graphical Output

Checking the graphical output first can help the modeler find any obvious problems. The profile plot will reveal specific areas of concern, which the user may then focus on utilizing cross section plots.

7.3.2 Check Warning Messages

Almost all hydraulic models have warnings, notes, and cautions. Some are just informational notes and some are severe; however, modelers should make themselves familiar with each message by reading the model manuals. This familiarity gives the modeler justification for dismissing or taking seriously any messages produced by the model.

7.3.3 Check Tabular Output

After the graphical output and warning messages have been checked, tabular output can be viewed, focusing on the areas of concern picked up during the previous checks.

7.3.4 Check Velocity and Flow Distribution

Look at the velocities and flow percentage of the main channel and overbank areas. If the prototype is fairly uniform in the longitudinal direction, the velocities and flow distribution of the channel and overbanks also should be fairly consistent.

If the model shows severe changes occurring from cross section to cross section, there should be a physical reason for the changes. For instance, if channel velocity changes in the downstream direction from 6.0 fps to 2.0 fps to 7.0 fps and the channel geometry and slope are relatively uniform throughout the reach, this is not a reasonable model result.

It may be that where the channel velocity was 2.0, there was a large increase in overbank flow area, thus decreasing the channel discharge, resulting in a lower channel velocity. One may then consider if all the overbank areas should be available for conveyance. Expansion and contraction considerations should be applied to flow entering and leaving the overbank area and appropriate ineffective flow areas should be defined.

Another example would be if the percentage of discharge (say 60%) in one overbank changed to 10% in the next downstream cross section and the percentage in the channel stayed the same (say 20%). This means that the other overbank changed from 20% to 70% in the downstream direction. The modeler must judge if the reach length between the cross sections was sufficiently long to allow such a large transfer of water from one overbank, through the channel, and to the other overbank.

Appropriate adjustments would be to use the ineffective flow options or adjust the overbank "n" values. Again, remember to check the channel discharge and velocity in relation to the adjacent cross sections.

Consider also when a levee completely separates the overbanks and channels such that no water could interchange between the three major subsections. This means that the percentage of discharge in each subsection should not change from one cross section to another. This should be accomplished by use of the ineffective flow option and, to some extent, adjustments to the overbank "n" values.

8. References

Albridge, B.N. and Garrett, J.M. (1973). "Roughness Coefficients for Stream Channels in Arizona," *Unnumbered Open-File Report*, U.S. Geological Survey, 87 p.

Arcement, G.T., and Schneider, V.R. (1984). "Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Floodplains," *Report No. FHWA-TS-84-204*, Federal Highway Administration.

Barnes, H.H., Jr. (1967). "Roughness Characteristics of Natural Channels," *USGS Water Supply Paper 1849*, Washington, DC.

Chow, V.T. (1959). *Open Channel Hydraulics*, McGraw-Hill, New York, New York.

Cowan, W.L., (1956). "Estimating Hydraulic Roughness Coefficients," *Agricultural Engineering*, Vol. 37, No. 7.

Cunge, J.A., Holly, F.M., and Verwey, A. (1980). *Practical Aspects of Computational River Hydraulics*, Pittman Publishing, Inc., Marshfield, MA.

Davidian, J. (1994). "Computation of Water-surface Profiles in Open Channels," *Chapter A15, Book 3, Techniques of Water-Resources Investigations of the United States Geological Survey*, U.S. Geological Survey, Alexandria, VA.

FCDMC (2000). *Consultant Guidelines,* August 1, 2000.

FEMA (1995). "Flood Insurance Study Guidelines and Specifications for Study Contractors," *FEMA 37.*

FEMA (1990). "Certification Requirements for Simple Floodway Encroachments," Memorandum for FEMA Regional Directors from the Administrator of the Federal Insurance Administration dated October 31, 1990.

Hicks, D.M, and Mason, P.D. (1998). *Roughness Characteristics of New Zealand Rivers*, National Institute of Water and Atmospheric Research, Ltd., Christchurch, New Zealand.

Phillips, J.V. and Ingersoll, T.L. (1998). "Verification of Roughness Coefficients for Selected Natural and Constructed Stream Channels in Arizona," *Professional Paper 1584*, U.S. Geological Survey.

Phillips, J.V. McDoniel, D., Capesius, J.P., and Asquith, W. (1998). "Method to Estimate Effects of Flow-Induced Vegetation Changes on Channel Conveyances of Streams in Central Arizona," *Water Resources Investigations Report 98-4040*, U.S. Geological Survey, Tucson, Arizona.

Thomsen, B.W. and Hjalmarson, H.W. (1991). *Estimated Manning's Roughness Coefficients for Stream Channels and Floodplains in Maricopa County, Arizona*, Flood Control District of Maricopa County, Phoenix, Arizona.

U.S. Army Corps of Engineers (2001). "HEC-RAS, River Analysis System, Hydraulic Reference Manual," *CPD-69*, Hydraulic Engineering Center, Davis, CA.

U.S. Army Corps of Engineers (1995). *Flow Transitions in Bridge Backwater Analysis*, RD-42, Hydraulic Engineering Center, Davis, CA.

U.S. Army Corps of Engineers (1994). "Hydraulic Design of Flood Control Channels," *Engineer Manual 1110-2-1601*, Washington, DC.

Appendix A – Model Matrix

	Item	1	2	3	4	5	6	7	8	9	10	11	12
	Model	**Name**	**Developed by**	**Latest Version**	**Area of application**	**Operating platform**	**Extent of Usage**	**Computational Theory**	**Major limitations**	**Major strengths**	**Multiple profiles, different flows**	**Calculation of normal and critical depth, Froude number**	**Superelevation due to curved channel**
1-D Steady State Programs	**HEC-2**	Hydrologic Engineering Center-2	Hydrologic Engineering Center, USACE	Version 4.6.2 (May 1991)	Open channel flow in natural and constructed channels	DOS	Most commonly used program in the U.S from the 1970' through the 1990's by federal, state, local agencies and the private industry	Energy equation, gradually varied flow, standard-step method	Channels should have small slopes (less than 10 percent)	Wide usage, good documentation, stable code, the Corps' MENU-2 editor has useful utilities such as the EDIT-2 Program to check for errors in the input data deck, and SUMPO for generating custom output which can be imported into spreadsheet programs	Yes	Program defaults to critical depth if unable to converge.	No
	HEC-RAS	Hydrologic Engineering Center-River Analysis System	Hydrologic Engineering Center, USACE	Version 3.0 (March 2001)	Open channel flow in natural and constructed channels	Windows	Rapidly replacing HEC-2 as the program of choice for new studies	Energy and momentum equation, gradually varied flow, standard step method	Channels should have small slopes (less than 10 percent)	Wide usage, good documentation, excellent user interface, CHECK-RAS program provides stringent checking of model.	Yes	Yes	No
	HY8	HY8 Culvert Analysis	Federal Highway Administration	Version 6.1 (1999)	Culvert Analysis and Design	DOS	Used mainly for culvert analyses	Inlet and outlet control, tailwater rating curve, roadway overtopping	Primarily for culvert design. Single section analysis only. Does not always conserve Q.	Good for culvert analysis	N/A, discharge range used for culvert analysis	N/A	N/A
	QUICK-2	Quick-2	Federal Emergency Management Agency	Version 2.0	Floodplain analysis used for approximate studies	Windows	Used for approximate studies by FEMA and Study Contractors	Manning equation, normal depth method	Only for simple analysis of a cross-section or a small backwater model, very few features	Quick analysis	No, one flow at a time	Yes	N/A
	WinXSPRO	WinXSPro	U.S. Department of Agriculture Forest Service Stream Systems Technology Center / WEST Consultants	Version 2.1 B (July 1998)	Cross section analyzer, includes sediment transport analysis	Windows	Used mainly by the U.S. Forest Service	Thorne & Zevenbergen's, Nelson et al. method, Manning *n* normal depth, Jarret's equation for Manning *n*	Cannot calculate expansion or contraction losses, cannot calculate backwater effects. Single section analysis only.	Performs detailed analysis of a cross section. Can overlay cross-sections over time. Extensive bridge computations.	Computes flow for multiple stages. Profiles N/A.	Normal depth, Froude # yes. No critical depth.	N/A
	WSPRO	Water Surface Profiles	Federal Highway Administration	V061698 (June 1998)	Floodplain flow in rivers and streams, constructed channels	DOS, Windows Interface available through Scientific Software Group's SMS Interface	Used mainly by the Federal Highway Administration	Energy equation, Manning formula, standard step method	UNKNOWN	UNKNOWN	Yes	Can compute normal or critical elevation	No
	WSP2	Water Surface Profile-2	Natural Resource Conservation Service, USDA, formerly the Soil Conservation Service (SCS)	Version 2.0 (October 1993)	Floodplain flow in rivers and streams, constructed channels	DOS	Used very widely in the 70's and 80's for federal flood insurance studies conducted by the SCS, currently not in widespread use and not actively supported by the NRCS	Energy equation, gradually varied flow, standard-step method	Will no longer be supported, cross-sections are coded looking upstream left to right, the elevations of the ends of the cross-section need to be the same	UNKNOWN	UNKNOWN	UNKNOWN	UNKNOWN
	WSPG	Water Surface Pressure Gradient	Los Angeles County Flood Control District	Version 4.0 (1992)	Open channel and / or closed conduit systems	DOS	Used widely in Southern California for storm drain analyses involving pressure flow	Energy equation, standard step method	Restricted to one n-value for entire cross section.	In addition to open channel flow, can model pressure conduit flow in the same system.	No	Normal and critical depth, Yes. Froude number no.	No
	WSPGW	Water Surface Pressure Gradient for Windows	Civildesign Corporation (modified version of Los Angeles County Flood Control Districts's WSPG)	Version 12.96 (October 2000)	Open channel and/or closed conduit systems	Windows or DOS Versions	Used widely in California for storm drain analyses involving pressure flow	Energy equation, standard step method	Program will not calculate water surface profile when the friction slope is 1 (100 percent) or greater, critical depth cannot exceed 100 feet, vertical drops in invert elevations are not allowed	In addition to open channel flow, can model pressure conduit flow in the same system.	Yes	Yes	Yes
1-D Unsteady State Programs	**Advanced ICPR**	Advanced Interconnected Pond Routing Model	Streamline Technologies, Inc.	Version 2.20 (October 2000)	Stormwater management	DOS	UNKNOWN	Link-node concept, solves full dynamic equations (St. Venant), finite difference scheme.	UNKNOWN	UNKNOWN	Yes	Normal & Critical Depth Yes. Froude Number no.	No
	EPA-SWMM (EXTRAN)	Environmental Protection Agency-Storm Water Management Model (Extended Transport Block)	Environmental Research Laboratory, USEPA	Version 4.31 (January 1997)	Mostly for use in complex storm sewer networks; can intergrate hydrologic data	DOS, Windows interfaces available through 3rd parties (see item #9 below)	Used mostly in the Northeastern U.S., popular among academia	Link-node concept, solves full dynamic equations (St. Venant), finite difference scheme, explicit solution	Extremely complex, more geared toward storm sewer systems than open channel systems	Very complete model, both open channel and pressure flow combined, very wide usage, long history, stable code. Interfaces with other SWMM modules. Windows interfaces available through MIKE-SWMM (Danish Hydraulic Institute) or XP-SWMM, or Chi's PC-SWMM	UNKNOWN	UNKNOWN	UNKNOWN
	FEQ & FEQUTL	Full Equations Model & Full Equations Utilities Model	U.S. Geological Survey	FEQ V. 8.92, FEQUTL V. 4.86 (1997)	Floodplain flow in rivers and streams, constructed channels	DOS	Used by the USGS and by academia	Solves dynamic equations (St. Venant), finite difference scheme, Implicit Weighted 4-point scheme	UNKNOWN	Well documented, very complete open-channel unsteady flow model.	N/A	Yes	No
	HEC-UNET	Hydrologic Engineering Center-One-Dimensional Unsteady Flow Through a Full Network of Open Channels	Hydrologic Engineering Center, USACE	Version 3.2 (August 1997)	Floodplain flow in rivers and streams, constructed channels	DOS	Widely used for modeling subcritical flow in large rivers, has also been used on creeks and urban channels	Solves full dynamic equations (St. Venant), finite difference scheme, 4-point implicit solution	Program not very stable for channels with steep reaches or abrupt grade changes	Well tested, widely used, very complete open-channel unsteady flow model	N/A, hydrograph routing	No, program solves equations for only subcritical flow conditions	No
	MIKE-11	MIKE 11	Danish Hydraulic Institute, Denmark	Version 4.10 (1999)	Floodplain flow in rivers and streams, constructed channels..	Windows	Not widely used in the U.S., popular in Europe	Solves dynamic equations (St. Venant), finite difference scheme, 6-point implicit finite difference	High cost, requirement that consulting firms purchase software to peform studies	Excellent user interface, extensive options, good for complex networks, has good GIS interfacing., supports Digital Elevation Models (DEMs)	N/A, hydrograph routing	No for Normal and critical depth, yes for Froude number	No

Appendix A – Model Matrix

	Item	13	14	15	16	17	18	19	20	21	22	23	24
	Model	**Frictional loss factor**	**Automatic flow regime determination, subcritical / supercritical**	**Junction analysis (in pipes, manhole losses, etc)**	**Hydraulics of tributary streams and laterals**	**Bridge hydraulics**	**Bridge pier analysis**	**Culvert analysis, wall entrance, wall exit, multiple pipes, pressure / non-pressure flow**	**Closed storm drain hydraulics**	**Overbank flooding**	**Automatic split flow analysis**	**Error and warning messages**	**Hydraulic jump location**
1-D Steady State Programs	**HEC-2**	Manning *n*, any horizontal variation can be specified, various computatoinal options	No, program defaults to critical depth in cases where flow passes from one regime to another, it is necessary to compute the profile twice, alternately assuming subcritical and supercrtical flow	N/A	Yes	Low flow, pressure flow, and weir flow	Yes	Yes, but program cannot model multiple shapes	No	Yes	Yes, using the split flow option	Yes	No
	HEC-RAS	Manning *n*, or computation of *n* via input of equivalent roughness *k*; any horizontal variation can be specified, various computational options	Yes, if "mixed flow" option selected	N/A	Yes, user specified flow distribution	Low flow, pressure, and weir flow	Yes, various options	Yes, but program cannot model multiple shapes	No	Yes	No	Yes	Yes, if "mixed flow" option selected
	HY8	Yes	N/A	N/A	N/A	N/A	N/A	Culvert analysis yes, others N/A	N/A	N/A	N/A	No	No
	QUICK-2	Yes	N/A	N/A	N/A	N/A	N/A	N/A	N/A	Yes	N/A	UNKNOWN	N/A
	WinXSPRO	Multiple *n* values, other methods using particle size, cross-section characteristics	N/A	N/A	N/A	N/A	N/A	N/A	N/A	Yes, limited to a single cross section	N/A	Yes	N/A
	WSPRO	Allows for both horizontal and vertical variation in Manning *n* values.	No	N/A	No	Low flow, pressure flow, weir flow	Yes	Yes, program can model multiple shapes	No	Yes	No	Yes	No
	WSP2	Fixed Manning *n* for channel and overbank segments	UNKNOWN	UNKNOWN	Yes	UNKNOWN	UNKNOWN	UNKNOWN	UNKNOWN	UNKNOWN	UNKNOWN	UNKNOWN	UNKNOWN
	WSPG	Manning *n*, only one n value can be used for entire cross section	Yes	UNKNOWN	UNKNOWN	Yes	Yes	Culvert analysis very limited, wall entrance and exit yes, multiple pipes no, pressure flow no.	UNKNOWN	UNKNOWN	No	UNKNOWN	Approximate
	WSPGW	Manning *n*, only one n value can be used for entire cross section	Yes	Yes	Only at junction entrance for force analysis of main stream	Low flow and/or pressure flow	Yes, pressure and momentum	Yes, program can model multiple shapes	Yes	No	No	Yes	Yes
1-D Unsteady State Programs	**Advanced ICPR**	Manning *n*, any variation between cross section points can be specified	Yes	Yes	Yes	Yes, using WSPR routines to simulate	Yes, using WSPR routines to simulate	Yes	Yes	UNKNOWN	Yes	Yes	No
	EPA-SWMM (EXTRAN)	Manning *n*, up to one value each for channel, left overbank, right overbank for each section	UNKNOWN	Headloss at manholes not calculated. Expansion and contraction losses not calculated.	Yes	No	No	No	Yes	UNKNOWN	Yes	Yes	UNKNOWN
	FEQ & FEQUTL	Manning *n*, any horizontal variation can be specified, various computatoinal options	UNKNOWN	No	Yes	Yes, uses WSPRO routines to simulate	Yes, uses WSPRO routines to simulate	Yes	Yes	Yes	Yes	Yes	No
	HEC-UNET	Separate overbank Manning *n* values	No	N/A	Yes, full network simulation	Low flow, pressure flow, and weir flow	Yes	Yes, multiple shapes can be modeled but limited to five openings	No	Yes, option to model overbanks as a series of interconnected storage areas	Yes, can model with lateral spillway connecting to storage areas in overbank	Yes, but limited, troubleshooting is difficult	No
	MIKE-11	Manning *n* or Chezy *C*	Yes	UNKNOWN	Yes	No, slated for future version Spring 2001	No, slated for future version Spring 2001	Yes, but program cannot model multiple shapes	No	Yes	Yes	Yes	Yes

Appendix A – Model Matrix

	Item	25	26	27	28	29	30
	Model	Friction loss calculation	Graphics	Metric / English Units (input and output)	Automated interpolated cross-sections	Floodway computation	FEMA Approved for National Flood Insurance Program Usage
1-D Steady State Programs	HEC-2	Yes, 4 options	Yes, the PLOT2 program within the Corp's MENU2 editor can be used	Yes	Yes, based on maximum specified change in velocity head	Yes, various options	Yes
	HEC-RAS	Yes, 4 options	Yes, several options	Yes, option to convert the input file to either units, no output option unless data is first converted by program	Yes, program requires user to specify location of interpolated cross sections	Yes, various options	Yes
	HY8	N/A	No	Yes	N/A	N/A	Yes, for culverts only
	QUICK-2	N/A	Yes, shows cross section plots	English Units only	N/A	N/A	Intended for use in areas studied by approximate methods (Zone A) only. May be used to develop water-surface elevations at one cross section or a series of cross sections.
	WinXSPRO	Yes, 4 options	Yes, several options	Yes	N/A	N/A	No
	WSPRO	Yes, 4 options	Some graphics with FHWA's HYDRAIN interface, more sophisticated graphics with Scientific Software Group's SMS interface	Yes	No	Yes	Yes
	WSP2	UNKNOWN	UNKNOWN	UNKNOWN	UNKNOWN	Yes, when used together with NRCS's FLDWAY program	No, NRCS is no longer supporting the program.
	WSPG	UNKNOWN	No	English units only	UNKNOWN	No	Yes, in Los Angeles and adjacent counties.
	WSPGW	Yes, Arithmetic mean	Yes, has a .DXF output option which can be imported into CAD programs	Yes, 4 input options with flow rates as small as gallons per minute or liters per second, output option in Metric or English	Yes, automatic when velocity head changes by 10% in reaches	No	Yes
1-D Unsteady State Programs	Advanced ICPR	Yes, 4 options	Yes	Yes	UNKNOWN	No	Yes
	EPA-SWMM (EXTRAN)	Yes	No, graphics available through 3rd party interfaces MIKE-SWMM, XP-SWMM, PC-SWMM	Yes	No	No	Yes, EPA-SWMM version only. Calibration or verification to the actual flood events highly recommended. Structural loss calculations unavailable and must be accommodated via roughness factor manipulation. Preferably, for NFIP purposes, head losses at bridges should be verified using WSPRO; losses at culverts should be verified using the U.S. Geological Survey's six equations for culvert analysis. Losses at storm sewer junctions should also be verified with separate calculations
	FEQ & FEQUTL	Yes	No	Yes	Yes	Yes	Yes, except Type 5 culvert flow computations of FEQUTL need verification with results obtained using methodology or models accepted for NFIP use.
	HEC-UNET	Yes	No, however output can be exported to Corp's Data Storage System (DSS) for display	English units only	Yes, but actual cross sections are not interpolated, instead additional computational nodes are inserted	Yes, but not FEMA approved, currently under review	Yes. Calibration or verification to the actual flood events highly recommended. Structural loss calculations unavailable and must be accommodated via roughness factor manipulation. Floodway concept formulation unavailable. Preferably, for NFIP purposes, head losses at bridges should be verified using WSPRO; losses at culverts should be verified using the U.S. Geological Survey's six equations for culvert analysis. Losses at storm sewer junctions should also be verified with separate calculations; contact FEMA for guidance with these calculations.
	MIKE-11	Yes	Yes.	Yes	Yes	No, slated for future version Spring 2001	Yes, but bridge flow computations need verification with results obtained using methodologies or models accepted for NFIP usage. Calibration to actual flood events required.

Appendix B – Hydraulic Models Accepted by FEMA for NFIP Usage

(Nationally Accepted Models - shown in descending order of approximate usage)
Effective: January 11, 2002

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
Hydraulic Models: Determination of Water-Surface Elevations for Riverine Analysis				
One-dimensional Steady Flow Models	HEC-RAS 2.2 (September 1998)	U.S. Army Corps of Engineers	Water Resources Support Center Corps of Engineers Hydrologic Engineering Center (HEC) 609 Second Street Davis, CA 95616-4687 http://www.hec.usace.army.mil/	A HEC-2 file can be imported into HEC-RAS; the user must change the conveyance computations in HEC-RAS and make the necessary modifications to the bridge modeling before running HEC-RAS to duplicate the results obtained using HEC-2. The use of HEC-RAS for restudying a stream previously studied using HEC-2 is encouraged, as long as one of the following conditions is met: 1) the entire stream is rerun using HEC-RAS; or 2) the stream reach remodeled using HEC-RAS is hydraulically independent from the rest of the stream. The WSPRO bridge analysis is recommended for constricted floodplains under subcritical flow conditions. In addition, HEC-RAS version 2.2 that performs the steady flow water-surface profile calculations (SNET) has been updated to version 2.2.1; it should be used for NFIP studies.
	HEC-RAS 3.0.1	U.S. Army Corps of Engineers	Water Resources Support Center Corps of Engineers Hydrologic Engineering Center 609 Second Street Davis, CA 95616-4687	Under rare circumstances, for bridges with low flow, and weir flow on the overbanks, HEC-RAS 3.0.1 may not be able to balance the flow using weir flow equation and low flow bridge analysis methods. HEC-RAS 3.0.1 will then use the energy method, and the computed energy grade elevations and water-surface elevations may be on the high side.
	HEC-2 4.6.2[2] (May 1991)	US Army Corps of Engineers	Water Resources Support Center[3] Corps of Engineers Hydrologic Engineering Center 609 Second Street Davis, CA 95616-4687	Includes culvert analysis and floodway options.
	WSPRO (June 1988 and up)	US Geological Survey, Federal Highway Administration (FHWA)	Federal Highway Administration (FHWA) web page at: http://www.fhwa.dot.gov/bridge/hyddescr.htm	Floodway option is available in June 1998 version. 1988 version is available on the USGS web page at: http://water.usgs.gov/software/surface_water.html
	FLDWY (May 1989)	US Department of Agriculture, Natural Resources Conservation Service	US Department of Commerce National Technical Information Service 5285 Port Royal Road Springfield, VA 22161	Determines the encroachment stations from equal conveyance reduction method; used in conjunction with WSP2. Encroachment stations developed using this model must be re-entered in WSP2 model to properly develop floodway.
	QUICK-2 1.0 and up (January 1995)	FEMA	Federal Emergency Management Agency Hazard Identification Branch Mitigation Directorate 500 C Street, SW Washington, DC 20472	Intended for use in areas studied by approximate methods (Zone A) only. May be used to develop water-surface elevations at one cross section or a series of cross sections. May not be used to develop a floodway.
	HY8 4.1 and up (November 1992)	US Department of Transportation, Federal Highway Administration (FHWA)	Federal Highway Administration (FHWA) web page at: http://www.fhwa.dot.gov/bridge/hyddescr.htm	Computes water-surface elevations for flow through multiple parallel culverts and over the road embankment. Software and related publication are available from Center for Microcomputers in Transportation (McTrans), University of Florida, 512 Weil Hall, Gainesville, FL 32611-6585; and on the web at: http://www-mctrans.ce.ufl.edu/

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
	WSPGW 12.96 (October 2000)	Los Angeles Flood Control District and Joseph E. Bonadiman & Associates, Inc.	Joseph E. Bonadiman & Associates, Inc. 588 West 6th Street San Bernardino, CA 92410 http://www.bonadiman.com	Windows version of WSPG. Computes water-surface profiles and pressure gradients for open channels and closed conduits. Can analyze multiple parallel pipes. Road overtopping cannot be computed. Open channels are analyzed using the standard step method but roughness coefficient can not vary across the channel. Overbank analyses cannot be done. Multiple parallel pipe analysis assumes equal distribution between pipes so pipes must be of similar material, geometry, slope, and inlet configuration. Floodway function is not available. Demo version available from: http://www.civildesign.com
One-dimensional Unsteady Flow Models	FEQ 8.92 and FEQUTL 4.68 (1997, both)	Delbert D. Franz, Linsley, Kraeger Associates; and Charles S. Melching, USGS	US Geological Survey 221 North Broadway Avenue Urbana, IL 61801 http://water.usgs.gov/software/feq.html and technical support available at http://www-il.usgs.gov/proj/feq/	The FEQ model is a computer program for the solution of full, dynamic equations of motion for one-dimensional unsteady flow in open channels and control structures. The hydraulic characteristics for the floodplain (including the channel, overbanks, and all control structures affecting the movement of flow) are computed by its companion program FEQUTL and used by the FEQ program. Calibration or verification to the actual flood events highly recommended. Type 5 culvert flow computations of FEQUTL need verification with results obtained using methodology or models accepted for NFIP use. Floodway concept formulation is unavailable.
	Advanced ICPR 2.20 (October 2000)	Streamline Technologies, Inc.	Streamline Technologies, Inc. 6961 University Boulevard Winter Park, FL 32792	Previous versions of the model are not acceptable. Calibration or verification to the actual flood events highly recommended. Floodway concept formulation unavailable.
	SWMM 4.30 (May 1994), and 4.31 (January 1997)	US Environmental Protection Agency and Oregon State University	Center for Exposure Assessment Modeling US Environmental Protection Agency Office of Research and Development Environmental Research Laboratory 960 College Station Road Athens, GA 30605-2720 http://www.epa.gov/ceampubl/softwdos.htm Department of Civil, Construction, and Environmental Engineering Oregon State University 202 Apperson Hall Corvallis, OR 97331-2302 http://www.ccee.orst.edu/swmm/ ftp://ftp.engr.orst.edu/pub/swmm/pc	Calibration or verification to the actual flood events highly recommended. Structural loss calculations unavailable and must be accommodated via roughness factor manipulation. Floodway concept formulation unavailable. Preferably, for NFIP purposes, head losses at bridges should be verified using WSPRO; losses at culverts should be verified using the US Geological Survey's six equations for culvert analysis. Losses at storm sewer junctions should also be verified with separate calculations; contact FEMA for guidance with these calculations. Supporting documentation for floodway calculations is availableat: http://www.fema.gov/mit/tsd/dl_swmm.htm.
	UNET 4.0 (April 2001)	US Army Corps of Engineers	Water Resources Support Center Corps of Engineers Hydrologic Engineering Center (HEC) 609 Second Street Davis, CA 95616-4687	Calibration or verification to the actual flood events highly recommended. Comparison of bridge and culvert modeling to other numerical models reveals significant differences in results; these differences may be investigated in the near future. Floodway option currently under review, not accepted for NFIP usage.
	FLDWAV (November 1998)	National Weather Service	Hydrologic Research Laboratory Office of Hydrology National Weather Service, NOAA 1345 East-West Highway Silver Spring, MD 20910	Includes all the features of DAMBRK and DWOPER plus additional capabilities. It is a computer programfor the solution of the fully dynamic equations of motion for one-dimensional flow in open channels and control structures. Floodway concept formulation is unavailable. Calibration to actual flood events required. This model has the capability to model sediment transport. Program is supported by NWS. Supporting documentation is available at: http://www.fema.gov/mit/tsd/dl_fdwv.htm

TYPE	PROGRAM	DEVELOPED BY	AVAILABLE FROM	COMMENTS
	MIKE 11 HD (June 1999)	DHI Water and Environment	DHI Inc. Eight Neshaminy Interplex Suite 219 Trevose, PA 19053	Hydrodynamic model for the solution of the fully dynamic equations of motion for one-dimensional flow in open channels and control structures. The floodplain can be modeled separately from the main channel. Bridge flow computations need verification with results obtained using methodologies or models accepted for NFIP usage. Calibration to actual flood events required. Floodway concept formulation is unavailable. This model has the capability to model sediment transport. The web page is at: http://www.dhi.dk
	FLO-2D v. 2000.11 (December 2000)	Jimmy S. O'Brien, Ph.D., P.E.	FLO-2D Software, Inc. Tetra Tech, ISG P.O. Box 66 Nutrioso, AZ 85932	Hydrodynamic model for the solution of the fully dynamic equations of motion for one-dimensional flow in open channels and two-dimensional flow in the floodplain. Bridge or culvert computations must be accomplished external to FLO-2D using methodologies or models accepted for NFIP usage. Calibration to actual flood events required. Floodway computation is unavailable.
Two-dimensional Steady/Unsteady Flow Models	TABS RMA2 v. 4.3 (October 1996) RMA4 v. 4.5 (July 2000)	US Army Corps of Engineers	Coastal Engineering Research Center Department of the Army Waterways Experiment Station Corps of Engineers 3909 Halls Ferry Road Vicksburg, MS 39180-6199	Limitations on split flows. Floodway concept formulation unavailable. More review anticipated for treatment of structures.
	FESWMS 2DH 1.1 and up (June 1995)	US Geological Survey	US Geological Survey National Center 12201 Sunrise Valley Drive Reston, VA 22092 http://water.usgs.gov/software/surface_water.html	Region 10 has conducted study in Oregon. Floodway concept formulation unavailable. This model has the capability to model sediment transport.
	FLO-2D v. 2000.11 (December 2000)	Jimmy S. O'Brien, Ph.D., P.E.	FLO-2D Software, Inc. Tetra Tech, ISG P.O. Box 66 Nutrioso, AZ 85932	Hydrodynamic model that has the capabilities of modeling unconfined flows, complex channels, sediment transport, and mud and debris flows. It can be used for alluvial fan modeling.
Floodway Analysis	SFD	US Army Corps of Engineers/FEMA	Federal Emergency Management Agency Hazard Identification Branch Mitigation Directorate 500 C Street, SW Washington, DC 20472	Simplified floodway procedure for streams with no regulatory floodway limits.
	PSUPRO	Pennsylvania State University/ US Army Corps of Engineers/FEMA	Federal Emergency Management Agency Hazard Identification Branch Mitigation Directorate 500 C Street, SW Washington, DC 20472	Encroachment analysis for streams with no regulatory floodway limits.

[2]The enhancement of these programs in editing and graphical presentation can be obtained from several private companies.
[3]Program is typically distributed by vendors and may not be available through HEC. A list of vendors may be obtained through HEC.

Appendix C – HEC-RAS Reviewer's Checklist

INPUT CHECKLIST

1. GEOMETRIC DATA

A. Review the *project limits* (limits of data collection). Is enough information (cross sections) gathered both upstream of and downstream of the study reach? For subcritical regime, make sure that the upstream project limit is at a distance where the water surface profile resulting from a channel modification converges with the existing conditions profile (to evaluate any upstream impacts due to project alternatives). The downstream limit should be far enough to prevent any user-defined boundary condition from affecting the results within the study reach. For supercritical regime, the roles of the upstream and downstream project limits are reversed.

B. Check the *river system schematic*. Are the various river reaches (for a dendritic river system) properly connected? Inspect the location of junctions. Are the flow directions correct? Check the location of flow splits and flow combinations in looped networks (if any).

C. Review the *cross section geometry*. Does it characterize locations of changes in discharge, slope, shape or roughness; locations where levees begin or end, at bridges, culverts, weirs or other control structures? Are the cross sections properly oriented (perpendicular to the anticipated flow lines, i.e. approximately perpendicular to the ground contour lines)? Review individual cross-section plots. Does a cross section extend across the entire floodplain? Is each end of the cross section higher than the anticipated maximum water surface elevation? Is the topography of the channel (bank stations) and floodplain accurately reflected in the geometry of the cross sections?

D. Review the *reach lengths* (distances between cross sections). Check if the channel reach lengths are correctly determined along the thalweg and the overbank reach lengths measured along the anticipated path of the center of mass of the overbank flow. Make sure that the cross section spacing properly reflects the stream size (? 5 times channel width), slope, uniformity of cross-section shape and the purpose of the study.

E. Review the *profile plots* of channel bed elevations and top of bank elevations for abrupt changes, adverse grade or other anomalies.

2. FLOW DATA

A What is the design *discharge* and how is it derived?

B Is there *existing discharge* data (hydrologic record) that may be more appropriate or required for regulatory purposes?

C Are there any *tributaries* at which a change in discharge might be expected?

D Are there *multiple discharges* (multiple profile run)? What return interval (event) does each discharge represent?

E What is the expected *flow regime*? Is there a possibility for mixed flow regime?

3. BOUNDARY CONDITIONS

A. What method is used to establish the *starting water surface elevation*: observed, slope-area, critical or other? Is this method appropriate based on available information on flow regime and topography (for subcritical flow, boundary conditions are necessary at the downstream project limit; for supercritical flow, boundary conditions are necessary at the upstream project limit; for mixed flow, boundary conditions are necessary at both project limits)?

B. If there is not a known starting water surface elevation, prepare a range of user-defined starting elevations to check the *sensitivity* of the results in the study reach.

4. ENERGY LOSS COEFFICIENTS

A. What *Manning's roughness* coefficients are used for the channel and overbank areas? Review available aerial and/or ground photography. Conduct a field reconnaissance. Are the coefficients realistic and representative of vegetation, seasonal change, channel irregularities, channel alignment, channel slope, stage and discharge, and bedforms? Is there a need to model more than three distinct zones within each cross section (left overbank, channel and right overbank)? Does aerial photography or field review indicate braided channels or other areas with the horizontal variability of roughness? Check if the observed water surface profile information (gaged data and high water marks) is available for the roughness calibration. Compare the adopted Manning coefficients to those used in other studies for similar stream conditions and/or those obtained from experimental data.

B. What *expansion and contraction coefficients* are used to evaluate transition losses? Are they representative of the changes in geometry between successive cross sections and flow regime? Do they include energy losses at bridges, culverts, weirs and other control structures? Make sure that the coefficients applied between two cross sections are specified as part of the data for the upstream cross section.

5. INEFFECTIVE FLOW AREAS

A. Are non-conveying, *flow separation* areas where the velocity in the downstream direction is close to zero (e.g. "shadow" areas outside the main flow conveyance zone approaching or exiting a bridge, culvert or other flow obstacle) modeled as ineffective?

B. Are *depressions* such as overbank excavations or low grounds where water ponds but is not actively being conveyed, represented as ineffective flow areas?

6. SPECIAL CONDITIONS

Based on review of the input data, note the existence or any indication of the possible existence of the following conditions for further investigation after reviewing the output:

A. Bridges, culverts, weirs and other control structures

B. Levees

C. Blocked obstructions

D. Distributary or alluvial fan conditions

E. Split and/or divided flow

F. Islands

OUTPUT CHECKLIST

1. KEY HYDRAULIC PARAMETERS

Check the following parameters for consistency and reasonableness:

A. Flow depth

B. Critical flow depth

C. Velocity

D. Velocity head

E. *Area*

F. *Top width*

G. *Invert slope*

H. *Energy slope*

These parameters should vary gradually between successive cross sections. Note any unusual variations and any extreme values that do not seem realistic or are inconsistent with known conditions regarding the stream reach.

2. FLOW CONSISTENCY

A Check the *streamwise variation* (from cross section to cross section) in the flow distribution between the channel and the left/right overbank. Does the amount of flow (discharge) in any one area vary dramatically from one cross section to the next?

B Check the *lateral distribution* of flow between the channel and the overbanks for each cross section. Does it seem reasonable (e.g. if majority of flow is in one overbank, is this what is expected based on the input review)?

3. ERROR AND WARNING MESSAGES

Review summary of errors, warnings and notes generated after each run. It is important to note that the user does not have to eliminate all the warning messages. However, it is up to the user to determine whether or not these warnings require additional action for the analysis.

Some common messages to look for include:

A. If there are consistent warning messages indicating profile *defaulting to critical depth*, consider modeling the alternative flow regime (subcritical vs. supercritical). Mixed flow regime should also be attempted.

B. Each cross section with the *energy equation could not be balanced* message (so that critical depth was assumed), requires further examination. This message is often an indication of unstable modeling (due to insufficient number of cross sections and data points, inconsistent flows, inaccuracy of roughness coefficients and energy losses), rather than critical flow depths. However, it is up to the user to determine whether critical depth is a legitimate answer which indicates minimum specific energy of the flow (e.g. at a transition from subcritical to supercritical flow, at a sudden constriction in subcritical flow, etc.).

C. If there are any *extended cross section* messages (these messages indicate the computed vertical floodplain limits exceed the limits of the cross section), consider obtaining additional ground points from the available topography to "close" the cross section and account for the additional flow area. Consider if flow would actually leave the main channel at this location (split flow situation).

D. In case of *divided flow* messages, it should be first determined whether the water can actually flow on both sides of the dividing land feature at the specified flow rate. If so, this usually requires separate modeling of divided reaches.

E. Messages indicating change in *velocity head* and/or *conveyance ratio* exceeding allowable limit imply that the flow areas are changing abruptly between cross sections and may call for additional cross sections or specification of ineffective flow limits.

F. Any *other* messages should be examined and either eliminated or justified.

4. SPECIAL CONDITIONS

Based on the foregoing review, determine whether the model input or output suggests any of the following special flow conditions:

A. *Bridges* and/or *culverts*? Are boundary cross sections properly located? Has the correct computational method been used (energy, momentum, Yarnell, pressure and/or weir)? Are ineffective flow limits properly specified? Check the pressure/low flow distribution in the output.

B. *Levees*? Is flow confined within levees allowing overbank flow only above the levee crest stage?

C. *Blocked obstructions*? Note that these elements decrease flow area and <u>add</u> wetted perimeter when the water comes in contact with them (unlike the ineffective flow limits).

D. *Distributary* or *alluvial fan* conditions? Does the output indicate consistent occurrence of flows diverging from a common path without rejoining downstream or do flow characteristics indicate a gradually expanding pattern of flow with little or no boundary definition? If so, a distributary type flow pattern may predominate, making one-dimensional modeling impractical or impossible.

E. *Split* and/or *divided flow*? Flow overtopping a divide as side weir flow? Have these areas been accounted for using split flow modeling or other approximation to account for lost flow?

F. *Islands*? Do the model results indicate isolated flood free areas within the floodplain? The occurrence of islands may indicate a flow pattern similar to split or divided flow where the two (or more) separate flow paths around the island must be modeled separately to accurately determine flow profiles.